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THE BROADWAY SERIES OF ENGINEERING HANDBOOKS

ELEMENTARY PRINCIPLES OF SURVEYING

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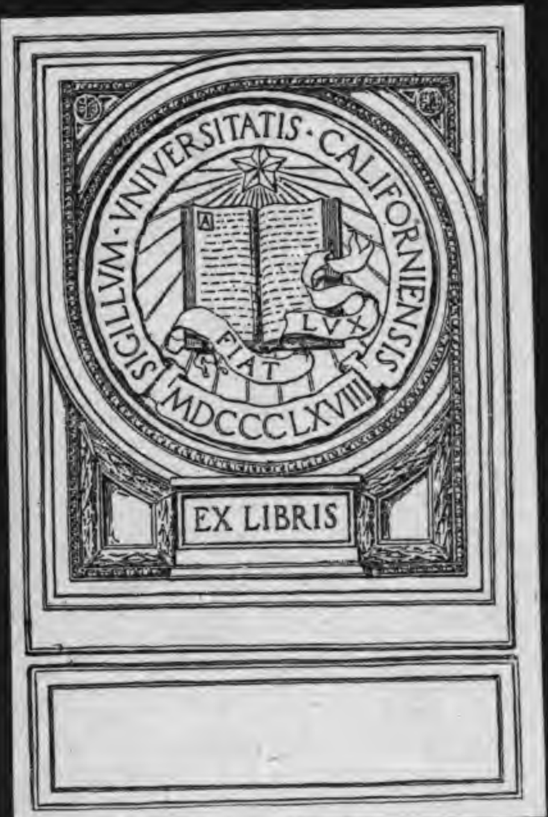
M. T. M. ORMSBY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ELEMENTARY PRINCIPLES OF SURVEYING

THE BROADWAY SERIES OF ENGINEERING HANDBOOKS
VOLUME XI

ELEMENTARY PRINCIPLES
OF
SURVEYING

A TEXTBOOK FOR THE USE OF STUDENTS,
ENGINEERS, ETC.

BY
M. T. M. ORMSBY
MEMBER OF THE IRISH INSTITUTION OF CIVIL ENGINEERS, MEMBER OF THE
ROYAL SANITARY INSTITUTE, LECTURER IN SURVEYING AT UNIVERSITY
COLLEGE, LONDON

WITH ONE HUNDRED AND THIRTY-EIGHT
DIAGRAMS AND ILLUSTRATIONS INCLUDING
FOUR FOLDING PLATES

LONDON
SCOTT, GREENWOOD & SON
8 BROADWAY, LUDGATE, E.C.
1914

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D. VAN NOSTRAND COMPANY
NEW YORK

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C. F.

PREFACE

In offering this little book to the student of Surveying, the writer feels that any apologies would be more or less out of place, for probably most teachers of the subject will agree that it is less well provided with the right kind of textbook than any other equally widely studied subject. Whether the appearance of this little work will help to improve matters in this respect or not is, however, a question which others must decide.

The author regrets that he is unable to claim even approximate *completeness* for the work, but this was manifestly impossible in a book of this size. His greatest difficulty in writing it was to decide *what to leave out.* It is probable that, on such a question, the judgment of any one person would differ considerably from time to time. It is therefore clearly hopeless to expect that one person's judgment at one time should meet with universal approval.

Nevertheless, it is hoped that the book may prove useful to some teachers and many students, and, above all, that no one will find in it a statement of supposed fact—(as distinct from an expression of opinion)—that it will be necessary for him to *unlearn* in his future study.

308534

The chief aim of the writer has been to supply the student who wished to acquire an elementary knowledge of Surveying with a book that would help him to acquire such knowledge on sound and accurate lines, and would, at the same time, help to fit him to study profitably some of the more complete manuals.

With this view, the chapters dealing with the more ordinary or elementary parts of the work, such as chain surveying and levelling, have been made comparatively full; and, within the scope of the work, those portions have been more particularly elaborated which are usually treated in a more or less scanty fashion.

Examples of such elaboration will be found in the arrangement of the lines for a chain survey (pp. 3 and 30), the temporary and permanent adjustments of the theodolite (pp. 64, 161, and 171), the adjustments of the instruments generally, the determination of earthwork quantities from sections and from contoured plans—(for example, by the method of mean sections, p. 131)—and so on.

The chapters dealing with the other parts of the work are necessarily more abridged, and have been written confessedly to some extent with an eye to the needs of examination candidates. The author has, however, tried to avoid writing anything which could be regarded as mere examination cramming, even in those chapters.

It is hoped that the exercises will prove useful to teachers, and the answers will be found to contain some hints which may be useful to students.

The author's experience as a teacher leads him to believe that nothing is more deadly dull than a chapter on instruments alone. He has therefore dealt with each instrument as the need for it arose. Many instruments have in consequence received no mention at all. The author must therefore close with the statement with which he began—one *had* to leave out something!

M. T. M. ORMSBY.

UNIVERSITY COLLEGE, LONDON,
April, 1914.

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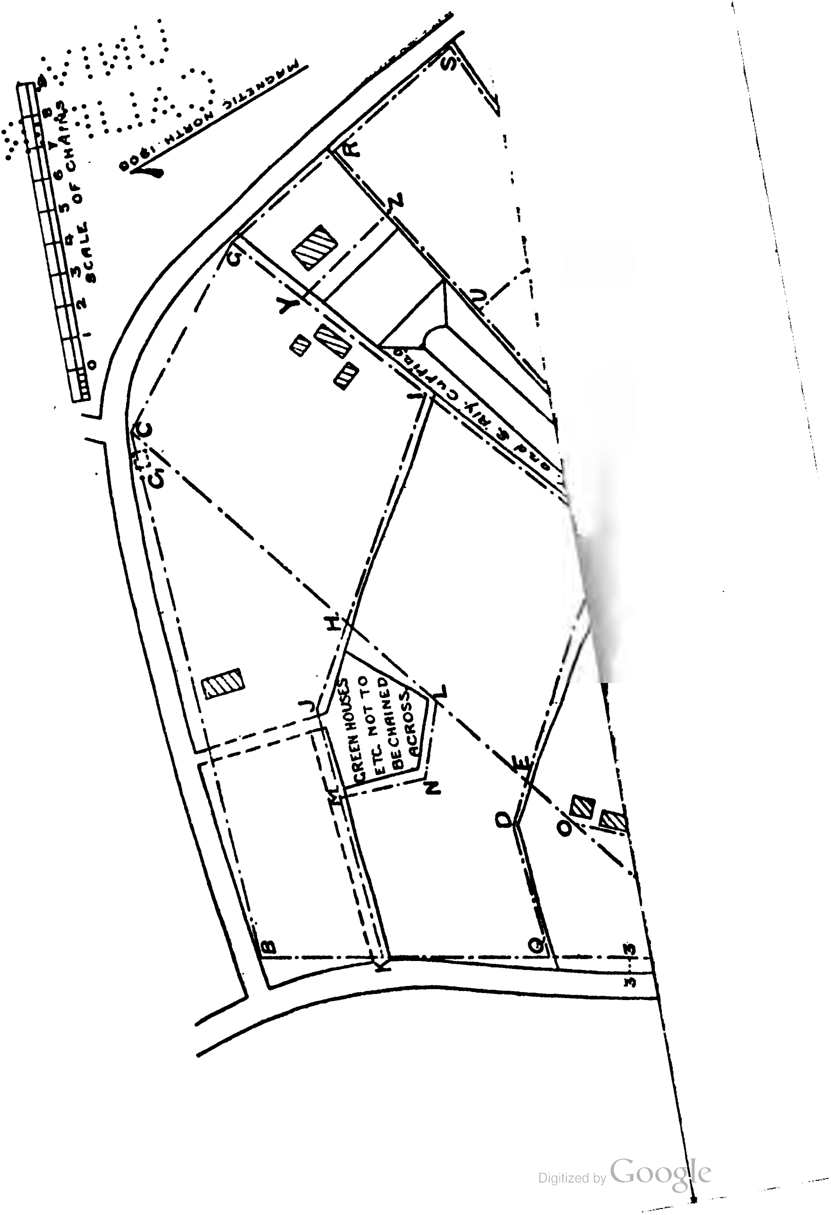

SCALE OF CHAINS
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GREEN HOUSES ETC. NOT TO BE CHAINED ACROSS
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UNIV. OF CALIFORNIA

CHAPTER I.

CHAIN SURVEYING.

Objects.—The objects for which any survey is undertaken may be placed, generally, under either or both of two main headings, viz.—

(*a*) The determination of the relative positions and dimensions of the various natural and artificial features on or near the earth's surface, as seen in *plan*.

(*b*) The determination of the relative *levels* or elevations of points lying on the natural surface of the earth, or on artificially constructed works of some kind.

The natural features which may have to be shown comprise the boundaries of seas, lakes, rivers, natural forests, mountains, etc., whilst artificial features include roads, railways, canals, buildings, boundary lines and fences, underground works for mines, and so on, either actually existing or projected at the time of the survey.

It is proposed in this chapter to deal with surveys of small extent only, and it will be assumed that levels are not required, so that we have to consider only the first of the above headings.

General Example.—Now suppose the sketch to represent a small estate, a plan or map of which is required.

Such small surveys are usually carried out by means of the chain and tape only, and are therefore called *Chain Surveys*, to distinguish them from larger surveys where angular measurements are necessary.

Method of Work.—The surveyor first walks over the ground and fixes points such as A, B, C, etc., called *stations*.

These are so arranged that the straight lines joining them will pass near to the boundaries or other features whose positions are to be fixed. These straight lines, such as AB, are called the *survey lines*.

Starting from one end of each line we then measure on the ground distances A1, A2, etc.; along the line, and at the points 1, 2, fixed in this way, we measure the perpendicular distances 1, 1; 2, 2, etc., from the survey line to the boundary. These perpendicular measurements are called *offsets*. The whole length of each line is measured up in this way on the ground.

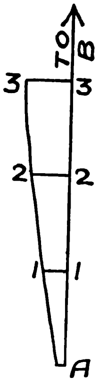


FIG. 2.

It will be obvious that, when these measurements have been taken, we can draw the line AB on paper (fig. 2), and mark off upon it the distances A1, A2, etc., and then by setting off the correct lengths of the perpendicular offsets, 1, 1; 2, 2, we can fix the positions of the points 1, 2, etc., on the boundary fence to any desired scale on the paper. All such points being joined we get the plan of the boundary line or fence.

Arranging the Lines.—Now suppose that the line AB (fig. 1) has been drawn on the paper. The other lines, BC, etc., have to be built up on it, each in its proper direction on the paper, with respect to the line AB. Now no angles are measured in the survey, so these other directions have to be fixed from the lengths of the lines alone. But a triangle is the only irregular figure which can be drawn from the lengths of its sides alone, hence the first condition to be borne in mind in arranging the lines is that we should arrange one or more *main triangles* as big as we can obtain them on the given piece of ground.

In the example ABC is the main triangle, and, having drawn AB, we would then take A as centre, and the measured length of AC as radius, and describe a circle to the proper scale. Then with B as centre, and the length of BC as radius, we describe another circle to cut this, thus fixing the point C on the paper.

The lines AC and BC can then be ruled in, and supposing that the rest of the lines are arranged as shown by the "dot and dash" lines on the figure, the stations Q, K, O, E, L and H can then be marked on the plan, their distances along the lines AB and AC having, of course, been measured on the ground when the survey was being made. AEF would be the next triangle to be drawn, F being fixed with A and E as centres and AF, EF as radii. It is not then essential that the rest of the lines should be arranged in triangles. Thus G would be fixed next with C and F as centres and CG and FG as radii, although the third side CF of the triangle was not measured.

To follow out the diagrams, after G is fixed we would produce EF to fix D and V. Then measure the length QD on paper. This should agree with the length measured on the ground, and so checks the work. Then fix I, rule in IH, and produce it to fix J. Then check JK, and so on. R is fixed from V and G as centres and checked by YZ; GR is produced to fix S. Then T is fixed from S and V as centres, and finally UT is measured on the drawing to check this part of the work.

Diagram of Lines.—The whole of this order of drawing (or *plotting* the lines, as it is called) must be thought out by the surveyor *whilst he is doing the survey.* To facilitate this a diagram is made in the field book showing the lines only. This is, of course, done on the field, and is not drawn to scale. The few main lines are first laid out, and shown on the diagram, the smaller lines being usually arranged as the work proceeds. As each line is arranged it is filled in on the diagram, in its proper place, the stations at the ends being marked with letters, as in the example, or in larger surveys with numbers.

Thus the surveyor can always study the order of plotting on his sketch diagram. The diagram, however, serves another purpose also. Each end station, as already stated, is fixed by the intersection of two circles. But two circles intersect in two points. Hence we must have some means of telling which is the correct one, and this will be obvious from the diagram. It also enables the draughtsman (who may not be the surveyor himself) to decide upon the most convenient order of plotting, which he does by studying the diagram.

The Field Book.—The field book in which the measurements are entered is shown in the annexed example (pp. 6 to 8), which is a set of notes for the north-west portion of the survey in fig. 1. The notes are, however, intended only to show how the measurements are put down. They are here abridged, and most of the *written* information is deleted to save space. The example is in part an actual survey, but it is in part fictitious, so that no name is given to it. Each page is ruled down the middle with two lines about half an inch apart. In entering the notes, the surveyor begins at the bottom of the page, enters first the name of the line to which that page refers, then makes a sketch showing how the boundaries, etc., are arranged in the *immediate vicinity* of the starting-point.

This sketch must not *cross* the central column. All objects on the *left* of the survey line (in the direction in which we are about to measure) are shown on the *left* side of the page, and vice versa, while any boundary or other line which *crosses* the survey line on the field (or *would* cross it if the survey line were produced) is shown coming in to one side of the central column, and starting again at the point directly opposite on the other side. Three fences crossing in this way are shown on the line AB, for example. It is not really necessary to give the points where they cross the line AB, because they will be fixed in position by offsets from the lines AW, QD, and KJ respectively. But it is usual to do so as a check. The central column is reserved for the distances measured along the chain line. Thus the fences cross the line AB at 5·32 and 10·48 chains from A, as will be seen from the field book.

SURVEY AT —— DATE ——

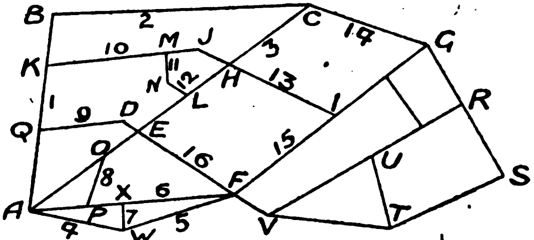
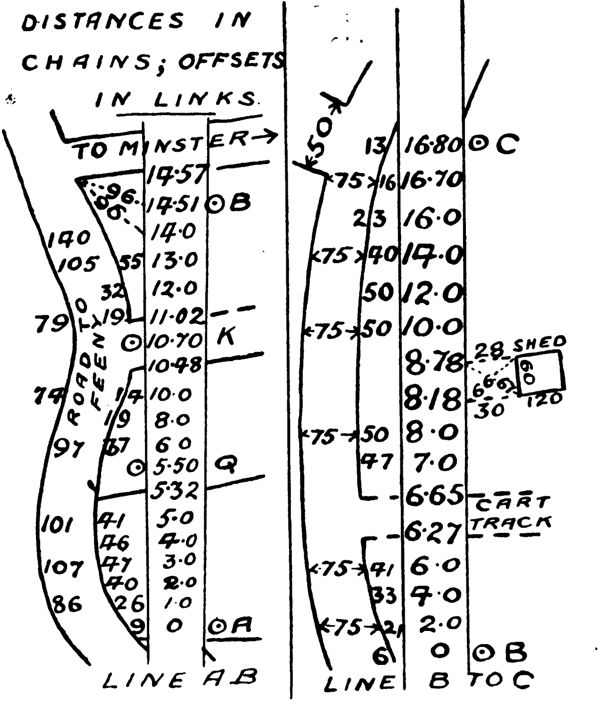
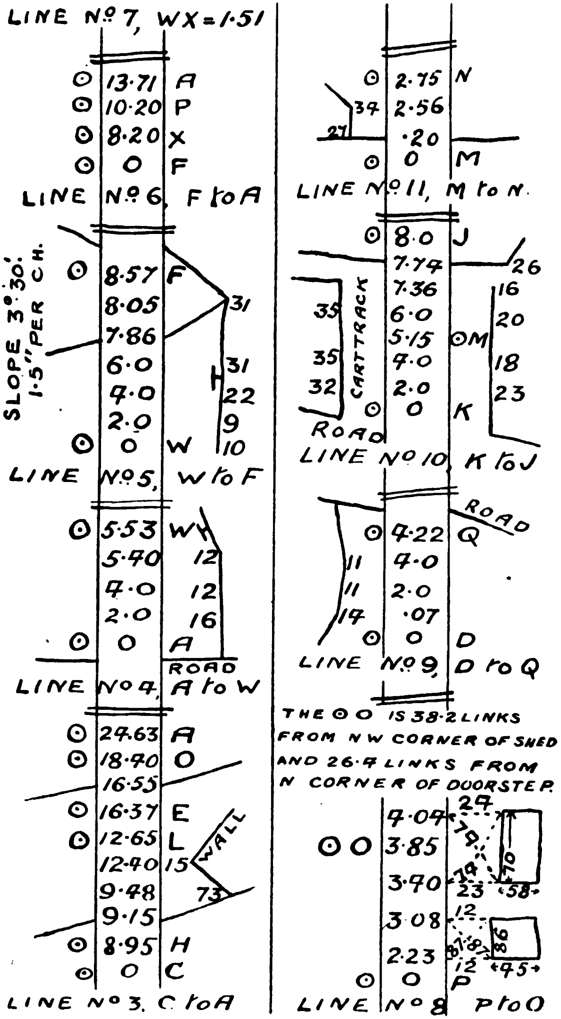
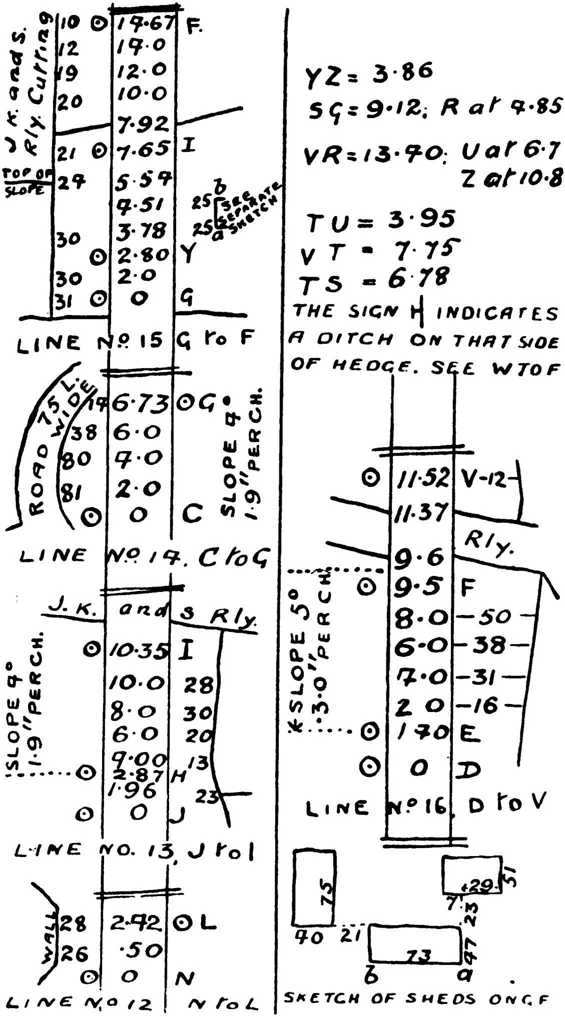


DIAGRAM OF LINES

DISTANCES IN CHAINS; OFFSETS IN LINKS.




LINE Nº 7, WX = 1·51
⊙ 13·71 A
⊙ 10·20 P
⊙ 8·20 X
⊙ 0 F
LINE Nº 6, F to A
SLOPE 3°30'
1·5" PER CH.
⊙ 8·57 F
8·05 31
7·86
6·0 31
4·0 22
2·0 9
⊙ 0 W 10
H
LINE Nº 5, W to F
⊙ 5·53 W
5·40 12
4·0 12
2·0 16
⊙ 0 A
ROAD
LINE Nº 4, A to W
⊙ 24·63 A
⊙ 18·40 O
16·55
⊙ 16·37 E
⊙ 12·65 L
12·40 15
WALL
9·48 73
9·15
⊙ 8·95 H
⊙ 0 C
LINE Nº 3, C to A
⊙ 2·75 N
34 2·56
27 ·20
⊙ 0 M
LINE Nº 11, M to N
⊙ 8·0 J
7·74 26
7·36 16
35 6·0 20
5·15 ⊙M
35 4·0 18
32 2·0 23
CART TRACK
⊙ 0 K
ROAD
LINE Nº 10, K to J
ROAD
⊙ 4·22 Q
11 4·0
11 2·0
14 ·07
⊙ 0 D
LINE Nº 9, D to Q
THE ⊙O IS 38·2 LINKS FROM N W CORNER OF SHED AND 26·4 LINKS FROM N CORNER OF DOORSTEP.
4·04 24
⊙O 3·85 79
3·40 74 70
23 58
3·08 12
2·23 81 87 86
12 45
⊙ 0 P
LINE Nº 8 P to O


J. K. and S. Rly. Cutting
10 ⊙ 19·67 F.
12 19·0
19 12·0
20 10·0
7·92
21 ⊙ 7·65 I
TOP OF SLOPE 24 5·59
4·51
25 b
SEE SEPARATE SKETCH
25 a
30 3·78
⊙ 2·80 Y
30 2·0
31 ⊙ 0 G
LINE No. 15 G to F
ROAD 75 L. WIDE
14 6·73 ⊙G
38 6·0
80 4·0
81 2·0
⊙ 0 C
SLOPE 4° 1·9" PERCH.
LINE No. 14. C to G
J. K. and S Rly.
⊙ 10·35 I
10·0 28
8·0 30
6·0 20
4·00 13
SLOPE 4° 1·9" PERCH.
⊙ 2·87 H
1·96 23
⊙ 0 J
LINE NO. 13, J to I
WALL
28 2·42 ⊙ L
26 ·50
⊙ 0 N
LINE No 12 N to L
YZ = 3·86
SG = 9·12; R at 4·85
VR = 13·70; U at 6·7
Z at 10·8
TU = 3·95
VT = 7·75
TS = 6·78
THE SIGN ⊢ INDICATES A DITCH ON THAT SIDE OF HEDGE. SEE WTOF
⊙ 11·52 V-12
11·37
Rly.
9·6
SLOPE 5° 3·0" PERCH.
⊙ 9·5 F
8·0 50
6·0 38
4·0 31
2 0 16
⊙ 1 40 E
⊙ 0 D
LINE No 16, D to V
75
29 31
7
23
40 21
73
47
b a
SKETCH OF SHEDS ON G F

The keeping of a neat book for these offsets is one of the most difficult pieces of work in surveying. The beginner must hold fast to the golden rule that *the sketch must not get ahead* of the measurements, but the two must be carried on together, line by line. Several different methods are in vogue for booking the offsets. Some of these are illustrated here. The beginner should carefully study these different methods of booking, and choose that which he prefers, referring to the diagram (fig. 1) as he studies the field book.

In the line AB, the offsets are simply written, each close to the boundary to which it refers.

They are all measured *from* the chain line. Thus the offsets at 5·0 tell us that at that distance from A the nearer side of the road is 41 links from the chain line, and the farther side 101 links. When, as in this case, more than one boundary is to be shown on the same side of the line, some surveyors prefer to measure the offset to the nearer boundary only, then the distance from that boundary to the next, and so on. This is exemplified in the line BC. Arrows are placed between the boundaries as shown, to make it clear that after the first offset the distances are from boundary to boundary. This practice is not recommended. It is less convenient for plotting than the system of running or continuous measure already described, and it is no more convenient in the field, even if the offsets are measured with a rod.

Some surveyors, again, when there are several offsets on one side of the line, prefer to write the offset figures sideways, the object being to make it clear to the draughtsman that they *are* in running measure, each one measured from the chain line, and *not* from boundary to boundary.

When the width of the road or other feature is fairly constant it may be simply measured and written down, as shown in the field book of the line CG, there being in this case no offsets to the second boundary.

In the field book of the line DV (p. 8) faint lines are drawn in the direction in which the offsets are measured, and the figure written in a break in the line, as shown. This is objectionable, as these lines are apt to be mistaken for fences branching off the main boundary and crossing the chain line.

Marking Stations.—As the stations are fixed they are marked on the ground by *ranging rods.* These consist of wooden rods, usually either six feet or ten links long, provided at one end with pointed iron shoes for sticking them in the ground. They are usually painted in two or three different colours, in lengths of 1 foot or 1 link as the case may be, so that they can be used as measuring rods for short offsets, etc. They are sometimes provided at the upper end with a small hook which can be used for dragging the end of the chain through a hedge or the like.

These are allowed to remain, marking the station until all the lines passing through that station have been measured, when they can be removed and used at other stations. But it is always advisable, unless the survey is a very small one indeed, to mark the positions of the stations, as soon as the ranging rods are removed, by driving in small pegs, by cutting out two triangular sods (if in grass land) having a common apex at the station point, or in any other way that may suggest itself. A good plan, if suitable objects are available, is to measure the distances of the station point from two near objects, or from two marks on such objects, and to note these measurements in the field book, describing carefully the points from which the measurements were taken.

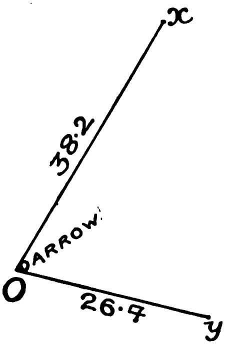


Fig. 3.

Fig. 4.

Thus in the field book of OP (p. 7) it is stated that O is 38·2 links from the north-west corner of shed, and 26·4 links from north corner of door-step, both measurements being taken from ground level.

To find O on the field, we add these measurements together: 38·2 + 26·4 = 64·6. An assistant then holds the end of a tape-line at the corner of the shed (point *x*) and another assistant holds 64·6 links on the tape at the corner of door-step (point *y*, fig. 3), while the surveyor holds an arrow at the point 38·2 links on the tape, and stretches both portions of the tape tight, whilst keeping the arrow at 38·2.

Such precautions save much time, should it be afterwards found necessary to check any part of the work or to take any additional measurements.

The Chain.—The surveying chain in most common use is shown in fig. 4. It is usually (in this country) either the Gunter's chain of 66 feet length, or the 100-foot chain. Abroad, lengths such as 25 metres are commonly used. But in any case it is divided into 100 parts, called links.

Thus in the Gunter's chain the length of 1 link is 0·66 foot, or 7·92 inches. When the terms "chains" and "links" are used with reference to surveying, in England, without qualification, it is usual to understand that Gunter's chains and links are intended.

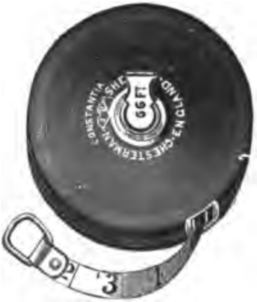


Fig. 5.

"Band" chains (fig. 5) consisting of flexible bands of steel, half an inch wide or less, are sometimes used instead of chains like those shown in fig. 4. They can be rolled up like an ordinary tape-line. In any case, the chain is provided at each end with a handle for pulling it taut, and each tenth link from either end is marked by a special brass tag. If the tag has *one* point, it is ten links from the nearer end; if *two* points, twenty links, and so on. This obviously facilitates counting the links when a measurement is required not at the end of the chain. But great care must be taken to see whether the portion being counted is more or less than half a chain, as the tag at sixty links from one end, for instance, is only forty links from the other end, and is marked with four points. The middle of the chain is marked by a round tag.

In the band chain, the tags are usually riveted to the band, and instead of points they usually have holes bored through them, one hole at ten links, and so on. The length of the chain is, in all cases, from the outside end of one handle to the outside end of the other.

Band and Link Chains Compared.—The special advantages of the band chain are that it is lighter than the link chain, and much less liable to stretch. With an ordinary chain of the form shown in fig. 4 there is an appreciable stretch for a comparatively small increase in the tension, so that the length varies with the pull given by the chainmen. Moreover it is liable in a short time to take on a permanent stretch, and hence must be tested from time to time. Any stretch is either allowed for by placing the arrow (which marks the end of the chain) short of the handle by the proper distance (according to the stretch) or, if circumstances permit, by having one or more of the small rings which connect the links removed, so as to correct the length. If more than one ring has to be removed, they should not be all taken from one part of the chain, but at equal intervals along the length.

The band chain, if properly used, stretches very little, but it is liable quickly to rust through if used on wet ground, and to break without notice. It is not easily repaired when this happens. The link chain is much less liable to break, and, if it does, it can easily be temporarily repaired with a piece of stout wire. On the whole the band chain is more suitable for very accurate work, but it should be carefully handled, and if it gets damp it should be well dried and oiled before being put away.

Gunter's and 100-foot Chains Compared.—The Gunter's chain was originally adopted because exactly 80 of these chains go to make 1 mile, or 10 to 1 furlong; moreover, in determining areas, 10 square chains make exactly 1 acre. Thus 1 acre = 4840 sq. yards = 43560 sq. feet, and 10 sq. chains = 10 × 66 × 66 sq. feet = 43560 sq. feet.

Hence this chain is still the most useful in cases where lengths are required in miles, as in road or railway surveys, or where the object of the survey is to determine the area of the ground in acres, roods, etc.

Moreover, the Standing Orders of Parliament require radii of curves on railways to be stated in Gunter's chains, in this country, and most of the tables in use for facilitating the calculation of the quantities of earthwork in cuttings and embankments, assume that the depths of these are known at points whose distance apart is known in Gunter's chains. But in surveys for such works as sewerage or water-supply schemes, or building plans, dock and harbour schemes, and so on, these advantages may be of no importance, and the 100-foot chain will generally be more convenient.

Arrows.—The end of each chain is usually marked by a pointed iron rod about 12 inches long, with a loop at one end to facilitate carrying. These are called arrows.

Method of Chaining.—Suppose any line (say AB, fig. 1, p. 1) is to be chained, starting from A. The work is done by two men, the one who goes first being called the *leader*, and the other the *follower*. The follower holds his end of the chain at A, taking care that the end of the handle is opposite to the middle of the ranging rod. He pushes the ranging rod slightly to one side or temporarily removes it altogether—and gets his eye as nearly as possible directly behind the station A. The leader, meanwhile, has taken his end of the chain towards B, taking *ten* arrows with him.

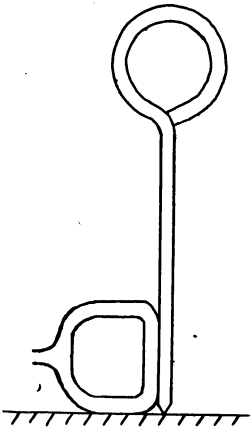


Fig. 6.

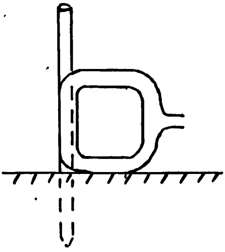


Fig. 7.

When the chain is tight, the leader faces the follower, who directs him, right or left, as the case may be, by a wave of the hand, until he (the leader) comes into line with the distant ranging rod at B. The leader then stoops down, or "squats," takes one arrow, and holds it vertically outside the handle at his end, the point of the arrow nearly level with the bottom of the handle, as shown in fig. 6. He then gives his end of the chain a chuck, at the same time pulling it towards him, so as to make the chain both tight and straight from end to end, finishing with the point of the arrow touching the ground. He then looks again towards the follower, and if the arrow is not exactly in line with B, he receives a second direction, and so on, until he finishes with the point of the arrow touching the ground, the chain tight and straight, the arrow in front of the handle, and exactly in line with B, as viewed by the follower at A. He then knocks or presses the arrow into the ground with the palm of his hand. The leader must, of course, be trained to keep his body on one side, holding his end of the chain out on one side or the other of himself, so that he does not obstruct the follower's view.

The chain is allowed to lie on the ground in position, until the surveyor has taken all the measurements he requires on that chain length. The surveyor then signals to the leader, and he takes up his end of the chain (but *not* the arrow) and proceeds. The follower replaces the ranging rod at A, and goes on to take up his position at the arrow left by the leader; he there holds the end of his handle in line with the *back* of the arrow, as shown by the sketch (fig. 7) (because the back of the arrow was the end of the first chain) and directs the leader as before, except that the leader can now get the direction approximately for himself by looking backwards from the arrow already fixed to the ranging rod at A. Thus the first preliminary direction is unnecessary, and the leader takes up the squatting position as soon as he has got himself into line. *When* the leader has fixed the next arrow the follower removes the one at his end, and keeps it. The follower must not previously remove the arrow, as he can see *over* it when directing. (He must not hold his head on one side of it.)

The beginner must study these instructions with minute care, as it will be necessary for him to train his chainman in most cases.

Changing Arrows.—It will be noticed that the number of arrows held by the follower at any time should tell how many completed chains have been "run" or measured. If the line is less than ten chains long, the surveyor should, when he arrives at the end of the line, count the arrows held by the follower and see that the number agrees with the number of complete chains entered in the book. He then hands the ten arrows to the leader to begin the next line.

If the line is more than ten chains long, it will be necessary to change over the arrows from follower to leader at the end of the tenth chain.

The practice of the writer is to instruct the chainmen to let the surveyor know when the arrows are to be changed, so that he may check his entry in the book.

26 This done, the follower hands *nine* arrows to the leader, leaving the one at the end of the tenth chain until after the eleventh has been fixed, when the remaining one is passed to the leader.

There is some risk in this procedure, as the follower may either forget to pass the tenth arrow over, or he may forget he has done so, and pass two over. The writer has not found the risk very great, but on account of it, and of some little loss of time, different methods are adopted by different surveyors. Some use only nine arrows, and some use eleven. The writer prefers ten arrows and the above method of changing, but the important point is that the chainmen should be trained on some definite system, and that the surveyor should, as far as possible, superintend the changing of the arrows.

It is of the utmost importance that no mistake should be made, as any neglect to observe and use the check which the arrows afford will very often lead to undiscovered errors in the chainage.

In some cases, where economy is of more importance than speed, the surveyor may himself act as follower.

Taking the Offsets.—The offsets are taken either with a light graduated rod or with a tape-line. The tape is decidedly more convenient than the rod, but linen tapes are liable to stretch considerably, and therefore for very accurate work it is necessary to test the tape frequently.

It is to be borne in mind, however, that exact accuracy in the offset tape is of *less* importance than in the chain, because each offset is measured independently and hence the error does not accumulate, whereas in chaining a line, say 20 chains long, the accumulated error will be twenty times the error in the chain. Graduated steel tapes for taking the offsets can be obtained, and they are best where high accuracy is required.

In any case, the direction of the offset is usually settled by eye only. Suppose that the offset is required from a point A on the chain line to a curved boundary CD (fig. 8). The surveyor stands at A with the body of the tape, while the offset man takes the end of the tape, and proceeds to the boundary. Both stoop so that the tape is stretched tight along the ground. The surveyor then looks to see if, in his opinion, the direction of the tape is perpendicular to that of the chain as it lies on the ground. If not, he signals the offset man right or left, as the case may require, until he arrives at the point D, where the tape AD appears to be perpendicular to the chain.

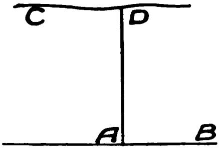


FIG. 8.

The surveyor then reads off both the chainage and the length of the offset, the offset man being trained, of course, to hold the end exactly on the boundary when he gets the final direction.

If the offset is required from a fixed point D on the boundary, say a corner, the only difference is that the offset man holds the end there, and the surveyor himself moves along the chain line, keeping the tape tight till the perpendicular direction is obtained as before.

It is not necessary to wind up the tape after each offset on good ground, but if it *is* necessary to do so, the writer has generally found it quicker to do it himself, unless there is much sketching to be done in the field book. In such cases a good offset man will walk towards the surveyor, gathering up the tape in loops as he goes, so as to do away with the necessity of winding.

It is more convenient for the offset man to keep the body of the tape, but this means that he must be trusted to read the offset correctly, and to call it out to the surveyor so distinctly as to avoid the possibility of a misunderstanding. The risk is too great. Even if read by an experienced observer, all calling out of figures from one observer to another is to be avoided, as it seldom fails to lead to errors. At the end of a day's practice the young surveyor will find that he can quite easily manage both the tape and his book without ever putting either of them into his pocket.

Size of Book.—The field books can be purchased with the two lines ruled down the middle of the page as recommended.

The size of the page is generally rather small, however. It should be of such a size that the book just fits in the pocket of an ordinary lounge coat. Messrs. Spon supply field books of Willesden paper, for use in wet weather, which are sometimes convenient.

The book may open either at the side of the page, as in any ordinary printed book, or it may open at the top or bottom of the page, as in the usual form of ruled field book. In the latter case, the beginner must take care, for the first page or two, to see which end he must take as the bottom of the page and which as the top, in order that the notes may be continuous from page to page.

Fixing Directions of Offsets.—If the offset be at all long, it may be necessary to lay out their directions more accurately than by eye. By chain and tape only, this may be done as follows: suppose that an offset is to be taken at A, on the chain line BAC (fig. 9). Mark two points B and C, such that AB and AC are the same length, say 30 links each. Then let one assistant hold the end of the tape at B, while another assistant holds the other end at C. Then assuming that the tape is 100 links long, the surveyor holds a ranging rod or arrow at the middle of the tape, that is the 50 link mark, and stretches both portions of the tape tight, moving so as to accomplish this without moving the arrow or rod away from the 50 link mark. He thus fixes a point D, such that CD = BD, and AC = AB. The direction AD will then be perpendicular to BAC, and the arrow or rod being stuck in at D the offset is measured in the direction AD.

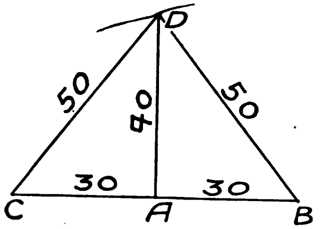


Fig. 9.

With the lengths suggested above, AB = 30 links and BD = 50 links. By a well-known property of right-angled triangles AD should then measure exactly 40 links, which affords a check on the work. Some surveyors make use of this property alone for laying out the perpendicular, without using a third point C. That is, make AB = 30 links. Then hold the end of the chain at B, and the 90 link point at A (note 50 + 40 = 90), while the surveyor holds the rod or arrow at the 50 link point and stretches both sides tight as before. In the opinion of the writer this is less accurate in practice than the above method, and it is little less troublesome. If there is no room for the lengths given, shorter lengths can be used, but of course they must be in the same proportion as the above if the check described is to be used.

It is not wise to depend upon equal stretching throughout the tape. That is, if we use the whole length of tape—for example—in fixing D, we should not assume that the 50 link mark is really the middle of the tape, but should bring the two ends together to find the true middle point. If the tape is incorrect, there may be some error when checking the length of AD.

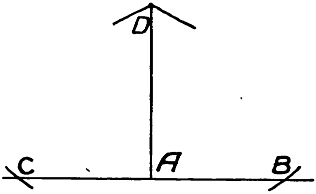


FIG. 10.

If the offset is required to a fixed point D, we have to fix the point A from which it is to be taken (fig. 10). For this purpose hold the end of the tape at D, and swing any convenient length round to cut the chain line at C and B. Mark these points. Then find A, the middle point of BC.

If the ground slopes, we might have, say, B and D about on the same level, and C much lower. This would lead to a wrong result, if the measurements were made along the ground, and we must make use of ranging rods at one or more of the points B, C, D. These are held vertical as well as can be judged by eye, and the tape is held at such a height up each rod as to bring the whole length of tape into a horizontal line or plane, this being also judged by eye.

The Optical Square.—The instrument now most commonly used for laying out the directions of offsets is called an *optical square.* It is made in many forms, but essentially each consists of two small mirrors, XY, XZ, at 45° to one another, as shown in plan in fig. 11, there being an open space above or below one or both mirrors, so that one can see straight ahead through this space at the same time as one looks into the mirror. If the eye be placed about at A, one can therefore see directly along the line AB, *over* the mirror XY. But by looking *in* the mirror, one sees objects situated in the direction of the point C. The rays from C follow the direction of the arrow-heads, and undergo double reflection from the two mirrors as shown, so that they finally enter the eye in the direction EA, as if they were coming from B. Now suppose there are ranging rods at B and C. Then we see directly the *upper* half of the rod at B, and we see by reflection in the mirror the *lower* half of the rod at C. These two will appear to be continu-

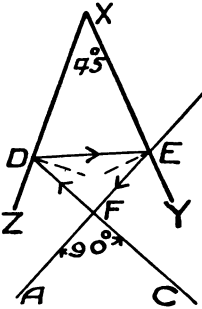


Fig. 11.

ous, and they are then said to be in coincidence. It is easy to prove, by the laws of reflection, that if ZXY = 45°, the angle AFC will be 90°.

To use the instrument, AB (fig. 12) is the chain line and let F be a point on it from which an offset is required, GCH the boundary to be fixed. The surveyor stands at F with the instrument to his eye, and looks straight through at the ranging rod at B. The offset man, meanwhile, has guessed the position C to which he must measure, and has gone there. The surveyor will probably be able to see him in the mirror, although probably not in exact coincidence with the ranging rod at B. He therefore signals left or right, as the case may require, and the offset man moves accordingly till he comes into coincidence with the ranging rod. The offset FC is then measured.

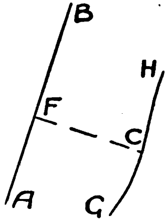


Fig. 12.

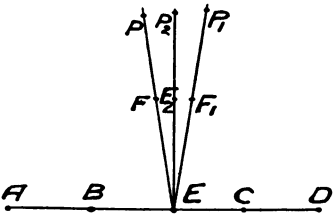


Fig. 13.

If the offset is required to a fixed point C, the difference is that the offset man stands at C, and the surveyor, with the instrument to his eye, walks along the chain line, until C and B appear to coincide. He will then be at the point F, where the offset is to be taken. By turning the instrument, objects on the left can be seen in the mirror XZ (fig. 11), whilst the rod B is seen above it.

Adjustment of Optical Square.—To test the adjustment of the optical square, range out four points in line with one another, A, B, C, D, say 50 feet apart. For this purpose, put in rods at any two of the points, say A and B. Then stand a few yards behind A, keeping in line with A and B, and signal to a man with a ranging rod to move right or left as the case may require, until he comes into line, first at D, then at C. Now take the instrument, and stand at some point E. Face A and B, and turn the instrument so as to see objects on the right in the mirror. Keep in line with the rods A and B, as seen directly, and direct the assistant at P right or left until a rod he holds appears to coincide with the rods at A and B. The rod P is then fixed. The assistant then moves in to F, and a second rod F is fixed in the same way, so that it appears to coincide with A, B, and P. Then turn round to face C and D, and turn the instrument so as to see objects on the *left*. Keep C and D in line with one another, and move till F and P also appear in line *with one another*. The instrument will then be over the same spot E as it was in the same position. If then the images of F and P, as seen in the mirror, coincide with C and D, the adjustment is perfect. But if, when C and D coincide and F and P also coincide with one another, the latter appear right or left of the former, the adjustment is faulty. In this case, have two more rods fixed, F_1 and P_1, so that their images coincide with those of C and D. Care must be taken when doing this to keep C and D in line with one another, and also F and P in line with one another. (See fig. 13.)

Then mark the points F_2, P_2, midway between F_1 and F and P_1 and P respectively, and place two of the rods at P_2 and F_2. Now face either A and B or C and D, and keep them in line. Move till F_2 and P_2 appear in line with one another, and adjust the screws at the back of the movable mirror till F_2 and P_2 coincide with A and B or C and D as the case may be.

The use of two ranging rods in each direction is the most satisfactory way of ensuring that the position of the instrument remains the same throughout the adjustment.

For actually adjusting the movable mirror, the beginner must study the arrangement of the screws at the back. He will probably find three screws, of which the two outer ones give motion in one direction, and the middle one in the opposite direction. First slacken, say, the outer ones about half a turn each, then tighten the middle one by the same amount, and look at the rods again. If they are nearer coincidence than before, the direction was the right one; if not, we turn the screws the opposite way, slackening now the middle one first, then tightening the outer ones. Proceed in this way, a little at a time, until the adjustment is perfect. Be careful to note that in all such cases the screws which are being slackened must be turned *first*. *Then* the opposing screw or screws must be tightened, so that there may be no looseness when the adjustment is complete. Different makers, however, provide different methods of adjustment, and the above arrangement will not be found in all cases, though the general principle will be much the same.

All Adjustments to be Tested.—The beginner must understand that even the best of makers can by no means be trusted to send out any kind of instrument always in perfect adjustment. The careful surveyor will always test his instruments when he receives them from the maker, and again from time to time, according to the amount of use they receive, as screws are apt to work loose during transit, etc.

The Cross-staff.—Another instrument for laying out the directions of offsets is the cross-staff, now less used than formerly. It consists of a cylindrical or octagonal box (fig. 14) with two pairs of slits at right angles to one another. The head can be rotated so as to bring one pair of opposite slits into coincidence with the chain

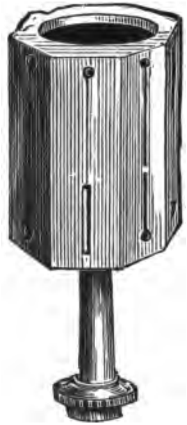


FIG. 14.

line, by looking through them at the ranging rod ahead or behind. By looking through the other pair of slits we can then fix the direction of the offset.

Sometimes additional slits are provided for laying out angles of 45°, and the head may also be in two parts, one of which is graduated in degrees, and the other carries a vernier, so that horizontal angles of any magnitude can be read or laid off.

Limiting Length of Offsets.—When the direction of the offset is laid out by the above methods, the only limit to its length is that fixed by questions of speed.

Long offsets take more time. Thus in fig. 15 instead of taking long offsets from the line AF to the boundary on the right, it would be quicker to simply measure AF and then run two additional lines, AW and WF, from which the offsets would be taken. The line WX should be measured as a check for the accurate plotting of the lines.

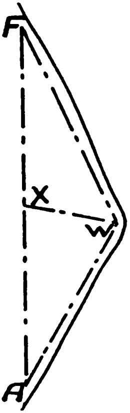


Fig. 15.

On the other hand, if only one or two long offsets be required to detached objects, it may be quicker to lay them out than to use additional lines.

It very seldom pays, however, to use longer offsets than the length of the tape line.

If the directions of the offsets are laid out by eye the length of offset is also limited by the question of speed, provided that the offset is to a point, such as X, on the boundary running nearly parallel to the chain line, because even if the offset be measured and plotted in different directions, as OX and OX_1, it will require either a very big angle between them or a very long length for OX and OX_1, in order to bring the point X_1 an appreciable distance away from the true plan of the boundary.

But if the offset be to the corner of a house, or to a corner in a fence, or any like point, then obviously the plan of the point X would be displaced on the paper by the whole amount XX_1. It is usual to consider that the offset should be so short that XX_1 will not exceed 0·01 inch in paper, this being regarded as the possible error in the plotting of any point by an ordinary draughtsman.

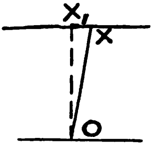


Fig. 16.

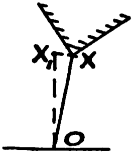


Fig. 17.

On this basis, and assuming that on fair ground the offset can be laid out by eye with an error in direction not exceeding 3°, the maximum length of offset to such a point will be about $n \div 5$, where n is the length represented by 1 inch on paper. Thus with a scale of 20 links to the inch, the surveyor should not lay out by eye (to corners or such points) longer offsets than $20 \div 5$, or 4 links, on the ground. If the scale is 100 feet to 1 inch, he can use $100 \div 5$, or 20 feet, and so on. Generally if a = angular error in direction of offset, l = measured length of offset in links, XX_1 = error in plotted point, n = number of links per inch in scale, then $XX_1 = l \sin \alpha$ links $= \dfrac{l \sin \alpha}{n}$ inches on paper. If we put $XX_1 = \cdot 01$ inch, we can find l from this formula, giving the limiting length of offset.

Taking Buildings, etc.—Now an optical square is not always available, and, especially if working to large scales, it may be inconvenient to take a line always within these distances of every corner. In such cases points, such as *a* (fig. 18), are fixed by taking two measurements to them from known points *e*, *f*, on the chain line. On paper, we mark the positions of *e* and *f* on the chain line, then take these as centres, and the lengths *ea*, *fa*, to scale, as radii, and describe arcs to meet, thus fixing the plan of *a*.

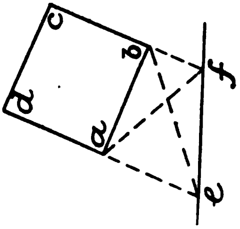


FIG. 18.

In the case of buildings, it is a good plan to fix the points *e* and *f* so that they are in line with the sides *ad* and *bc*. Then measure *ea*, *fa*, *fb*, and *eb*. Also measure the dimensions of the house. Examples of booking will be found in the field books referring to fig. 1, for example on the line OP, p. 7.

If the house is at all complicated, it may be difficult to get in all the dimensions in the space available in the field book. In such cases it is a good plan to fix only the two nearer corners, *a* and *b*, then give a reference to another page of the field book, where a dimensioned sketch of the building will be given, with the same two corners, *a* and *b*, lettered for identification. For an example, see the field book of GF, p. 8.

Other Forms of Field Book.—Some surveyors prefer to use a single line down the middle of the page, instead of the double line here shown and recommended.

In very complicated detailed surveys, as for instance in surveys for railway maintenance works, near stations and the like, this practice is not uncommon. But the whole field book becomes then rather of the nature of a dimensioned sketch, the chain line serving simply as a sort of base, along which the dimensions or distances are written, just as they are along any other line.

This system of booking has advantages in such cases, but the writer does not recommend it for ordinary work in open country.

Numbering Lines.—Many surveyors number the lines instead of the stations. In the opinion of the writer this leads to a more complicated field book without offering any compensating advantage, whereas simplicity in the field book should be one of the surveyor's chief aims. The only possible advantage known to the writer is that it may be easier to find the field book of any particular line, if it be at any time required to turn to it for reference—that is, assuming that they are measured in the same order as they are numbered. If this is considered worth troubling about, it is easy to number the lines on the diagram in the order in which they are measured, as shown on p. 6, so securing the above advantage at the same time as the greater simplicity due to the numbering or lettering of the stations.

Principles on which Lines are Arranged.—Returning now to the arrangement of the lines, it has been stated that the first object of the surveyor is (*a*) to obtain one or more main triangles of as large size as possible on the ground.

The principal other points to be attended to are as follows, and the student should study them in reference to fig. 1, p. 1:—

(*b*) There must be a line near enough to each boundary to avoid unduly long offsets.

(*c*) All the lines must be arranged so that they can be plotted by intersecting arcs from their lengths alone.

(*d*) The number of lines should be as small as possible, and, as far as possible, each line should be *useful*,—that is it should serve to directly fix some portions of the required boundaries, etc.

(*e*) There should be some check on the accuracy of each part of the chaining. That is, lines such as HI (fig. 1, p. 1) must be arranged to fit in between known points on other lines. If possible, these check lines should be otherwise useful for fixing boundaries, but, if not, they must be measured as checks alone, as in the case of UT.

(*f*) If any line, after being plotted, is to be produced to fix new stations, the lengths of the parts produced should be short in comparison with the part directly plotted. For example, see the produced parts FV and ED in the line EF.

(*g*) All intersections by which stations are fixed should be *well-conditioned.* That is, the arcs should meet at a good angle as shown in fig. 19. If they meet as in fig. 20, for example, the intersection is said to be ill-conditioned, and a very small error in either of the radii may cause a considerable displacement of the station.

(*h*) The crossing of hedges should be as much as possible avoided. Thus if we are surveying a line

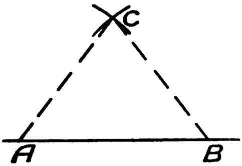


FIG. 19.

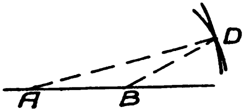


FIG. 20.

FK, and we wish lines to branch from it about in the directions KG and KH, we should first lay out, say, the line GL in about the direction in which we require it. Then, by means of a number of ranging rods close to the hedge, we can generally manage to see through the hedge from the other side, so as to prolong the line from L to K, and thus fix the position

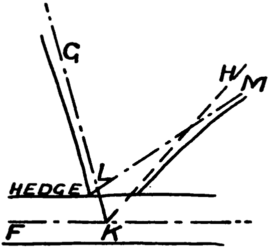


FIG. 21.

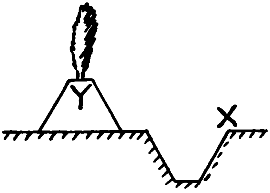


FIG. 22.

of station K. Then, along the line KG, take station L, so as to run the line LM instead of KH, thus avoiding a second crossing of the hedge.

Hedge and Ditch.—When a fence consists of a hedge with a ditch on one side, as shown in fig. 22, the offsets are usually taken to the centre of the hedge Y. But the true boundary is the edge X of the ditch, and an allowance must be made for this. Hence it is important that the young surveyor should look out for such cases and, if the ditch exists, that he should show in his field book on which side of the hedge it lies. (See lines AW and WF, p. 7.)

The allowance varies somewhat in different places, but it is usually 4 feet or 6 links according to the units in which the measurements are made.

Obstacles in Chain Surveying.—There are several kinds of obstacles to be met in carrying out a survey which render the task less easy of accomplishment. It is proposed to deal now with some of those which occur most frequently, and to suggest methods of overcoming them. But the first principle which the writer wishes the young surveyor to take thoroughly to heart is, that if accuracy is his chief object, he must be careful to arrange checks as far as possible for every part of the work. The *principles* of chain surveying (including the treatment of obstacles) are so easily grasped that the beginner is too apt to think that mistakes are almost impossible, to treat obstacles almost with contempt, and to neglect checks. Experience will teach him that such an attitude leads certainly to disaster, and that it is much easier to take a few extra measurements whilst the work is being done than to repeat the whole survey.

Sloping Ground.—The most common of such obstacles is sloping ground.

The distances to be shown on a plan are the *hori-*

zontal distances from point to point. Hence if a line has to be measured on sloping ground, a correction must be introduced. The first method of overcoming this is by "chaining in steps". For an *up* slope, assuming that A (fig. 23) is the end of a chain, the leader goes forward to B, and holds his handle on the ground. The follower raises his end of the chain up a ranging rod AD, until, as well as can be judged by eye, the chain is horizontal. The follower then holds tight, while the leader stretches the chain and inserts an arrow at B. Thus the *horizontal* distance DB is measured, and no further correction is necessary. The ranging rod may be held vertically by plumb-bob, or by eye alone. It is usual to work in half-chain lengths, as the whole chain is too heavy to be stretched horizontally at once. In this case, an arrow should be specially marked for the half-chain points, being passed back to the leader as the end of each complete chain is marked. The surveyor must see that when the slope is passed this arrow is returned to the leader or follower, according to the one from whom it was first taken. Some surveyors keep a special arrow for this work in addition to the ten ordinary arrows. On a down slope, the leader, of course, has the ranging rod.

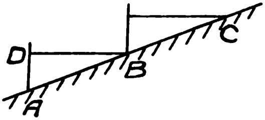


FIG. 23.

The method is only suitable for short slopes. On a slope of any length it would almost certainly lead to an appreciable accumulated error.

In the second method, if AB be the chain laid along the slope (fig. 24), it is desired to insert the arrow, not at B, but at C, so that AD may be one chain or a hundred links. For this purpose we measure the angle of slope, a. Then $AD = 100$ links and $AC = 100$ sec. a links, and $AB = 100$ links. Hence $BC = AC - AB = 100$ (sec. a $-$ 1) links.

This formula gives the amount BC by which the arrow must be placed by the leader *in advance of the end of each chain*, so that the horizontal distances from arrow to arrow may be one chain. It is called the *hypotenusal allowance*. Its value for various angles is given in the annexed table, which should be copied into the field book.

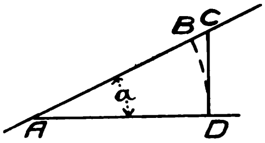


FIG. 24.

Angle of Slope.	Hypot. Allowance in Links.	In Inches per Gunter's Chain.	Inches per 100 ft. Chain.	Angle of Slope.	Hypot. Allowance in Links.	In Inches per Gunter's Chain.	Inches per 100 ft. Chain.
1°	·015	0·1	0·2	8°	·98	7·8	11·8
2°	·06	0·5	0·7	8°·30′	1·11	8·8	13·3
3°	·14	1·1	1·6	9°	1·25	9·9	15·0
4°	·24	1·9	2·9	9°·30′	1·39	11·0	16·7
5°	·38	3·0	4·6	10°	1·54	12·2	18·5
6°	·55	4·4	6·6	10°·30′	1·70	13·5	20·4
6°·30	·65	5·1	7·8	11°	1·87	14·8	22·5
7°	·75	5·9	9·0	11°·30′	2·05	16·2	24·6
7°·30′	·86	6·8	10·3	12°	2·23	17·7	26·8

The figures are given in inches (to the nearest tenth) per chain, as well as in links, in case it should be desired to apply the correction by means of a foot rule.

The Clinometer.—The angle of slope is most commonly measured by some form of clinometer. The most simple form consists of a semi-circle (say of white paper glued on to a thin board) graduated as shown (fig. 25). ACB is the diameter of the semicircle. At two points, A and B, on it, short pins are driven into the board, so that they project out perpendicularly from it. From the centre, C, is hung a small plumb-bob. To use it, a mark is made on a ranging rod, *at the same height above the ground as the observer's eye* (a strip of paper tied round the rod, with its top at the right height, will answer), and the leader is sent forward along the slope with it. The surveyor then aims the two pins, A, B, at the mark, holding the plane of the clinometer vertical, and his eye near A. He then clips the string of the plumb-bob with his thumb, lowers the instrument, and reads off the angle under the thread. The figure is drawn as if for an *up* slope, but the principle is the same whether the slope be up or down.

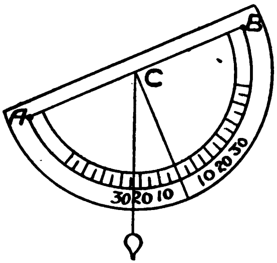


Fig. 25.

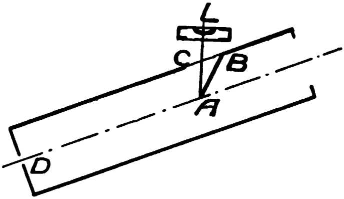


Fig. 26.

A more accurate clinometer is that shown diagrammatically in fig. 26. Essentially, it consists of a sighting tube, within which is fixed a reflector BA, as shown. The lower edge, A, of the reflector coincides with a diameter of the tube. There is an opening, BC, in the top of the tube, and above this is a level, tube L, which can be rotated, in a vertical plane, by a thumbscrew.

To use the instrument, we aim the tube at a mark on a ranging rod (at the same height above the ground as the observer's eye, in the same way as described above), using the small aperture D and the edge A of the reflector for taking aim. This brings the tube parallel to the slope, and the level L is then turned slowly by the thumbscrew. When it comes into the horizontal position, as shown, the bubble can be seen passing across in the reflector BA, and we adjust very carefully, so that the bubble appears central at the same time as the mark is covered by the edge A.

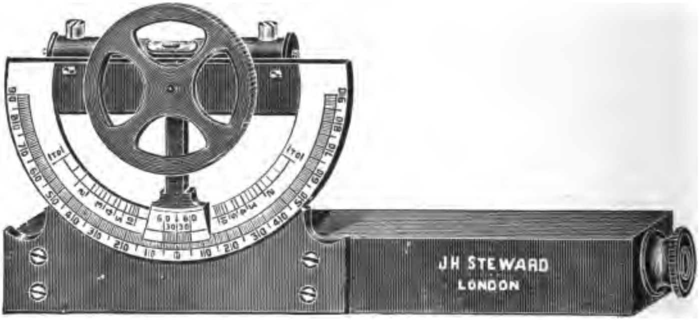


Fig. 27.

The instrument is then lowered, and the angle of slope read off by an index mark at the side.

The Abney Level.—The clinometer is similar in principle to the *Abney Level*, which is shown in figs. 27 and 28 in its ordinary form, and in the improved form as made by Mr. Steward, Strand, London.

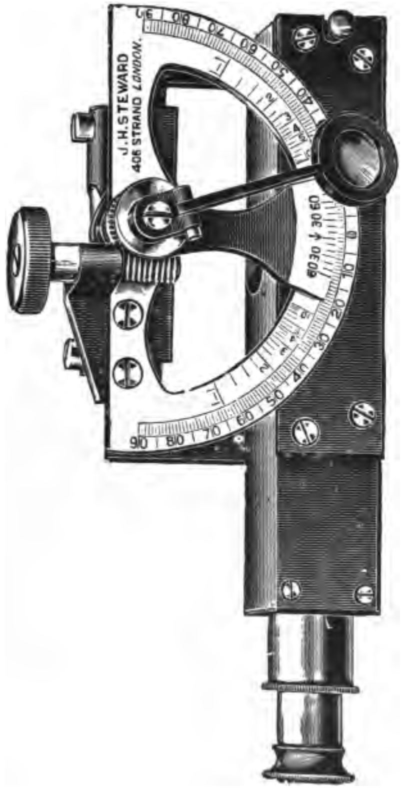


Fig. 28.

Adjustment of the Clinometer.—To test the adjustment of any form of clinometer, measure the angle of slope between any two points, say 100 to 300 feet apart, first *up* the slope, then *down*. Take two or three measurements each way, to see if the results are uniform. The result should be the same for the up as for the down slope. If not, *half* the difference gives the correction to be applied to all angles, and is called the *index error*. The sign of the correction depends upon which angle was the greater. For example:—

$$\left.\begin{array}{r}\text{up slope } 4^\circ{\cdot}30' \\ \text{down slope } 5^\circ{\cdot}0'\end{array}\right\} \begin{array}{l}\tfrac{1}{2}\text{ diff.} = 15', \\ \text{up slope too small.}\end{array}$$

Hence the index error is fifteen minutes, to be *added* to all angles of *elevation* (*up* slope angles) and *subtracted* from all angles of *depression*.

Many clinometers are graduated to give the hypotenusal allowance directly. In this case the index error cannot be used as here described, and the instrument must be corrected.

For this purpose, set the instrument to the *mean* of the two readings—(= 4°·45′ in the above example) —and adjust the bubble, a little at a time, by the screws which carry it, until when we look at the object with that setting, the bubble appears central.

Reason for Correcting for Slope on Field.—It would, of course, be possible to measure the whole line on the slope and correct for slope afterwards. But this is objectionable, because it would be necessary to calculate and apply the correction, not only for the whole line, but for every point at which an offset was taken. But by correcting at the end of each chain, as above, not only can the whole length of line be taken straight from the field book, but all offsets at the ends of chains can generally be plotted at once without any correction of distance. The correction on any *one* chain length is often so small that it would not be appreciable on paper. But in large scale work it may be necessary to correct the distances for any intermediate offsets. For this reason, wherever there is a slope it is desirable to note in the margin of the field book the angle of slope and the hypotenusal allowance. For example see field book of CG, p. 8. Thus with an offset at 1·63 chains, and hypotenusal allowance 0·8 link per chain, the draughtsman would work out ·63 × ·8 = ·5 link nearly, and the offset must be put in at 1·625 chains, instead of 1·63. There is no correction for the whole chain, for the reason already given.

Offsets, if on the slope, are generally measured by the method of *stepping* above described.

Ponds, Rivers, etc.—Other obstacles are such as ponds, rivers, quarries, etc., which we can *see* across. Here there is no difficulty about the *direction* of the line. First suppose that the distance, AB, across is less than one chain.

Then we chain up to A, on the near edge, then measure the distance AB across, and add to find the chainage of B. Thus suppose A = 11·60 chains, AB = 62 links. Then B = 12·22 chains. The follower then goes to B, and gets one arrow from the leader (so that the follower will now have *two* altogether, which corresponds to twelve chains) and holds the end of the *twenty-second* link at B (the chainage being 12·22), while the leader stretches the remainder of the chain to fix the third

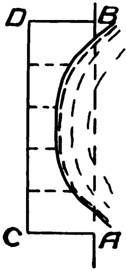


FIG. 29.

arrow, making thirteen chains. The surveyor must personally superintend this operation.

If the offsets to the boundary of the obstacle are required, perpendiculars AC, BD, may be laid out, of the same length. By eye will be sufficiently near if they are short. Measure along CD, and measure the offsets from that line, subtracting each one from the length AC or BD, so as to find the offset to be entered in the book. To avoid errors in distances, in the above example we should begin with the *sixtieth* link at C (the chainage at A being 11·60), and chain on from there. The chainage at D should check with the value already obtained for B.

AB should always be measured directly if possible, but, if longer than one chain, it may be necessary to rely on the measurements taken along CD. In this case the offsets AC, BD should always be laid out by optical square or by construction as already described.

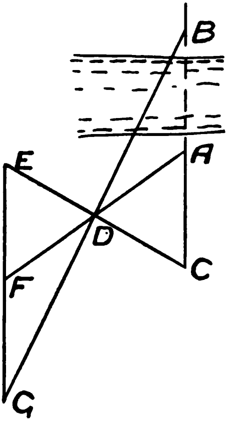


Fig. 30.

Another method, applicable to rivers as well as ponds, etc. (where the nature of the ground allows), is shown in fig. 30. It is assumed that AB is longer than one chain, as it would otherwise be measured directly.

Choose points C, A, on the chain line; also a point D, such that we may measure AD and CD, and produce them to make DF = AD, and DE = CD. Ranging rods are placed at

B, D, E, and F. The surveyor now walks along EF (keeping himself in line with the rods) until he also comes into line D and B. This fixes the point G. Then AB = FG, which latter can be measured. Chain up to A, calculate the chainage of B, and adjust the arrows as before.

Many other methods will suggest themselves. For instance, with a cross-staff set out AC at right angles to AB (fig. 31). Then by trial, using the 45° slits, find the point C such that ACB = 45°. Then AB = AC.

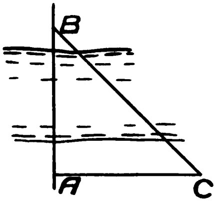


FIG. 31.

Or, if there is an instrument available for measuring horizontal angles, lay out AC in any direction for exact measurement, and measure also the angle BAC, ACB. Then calculate AB, by solving the triangle ABC. This involves the use of trigonometrical tables on the field, as the chainage should be adjusted at once, but it is probably the most accurate method in the hands of a careful arithmetician.

Houses, etc.—Obstacles such as houses, haystacks, and the like, are those which we *cannot* see across. Thus in fig. 32, DA is the chain line, and we require not only to find the distance AB, but also the position of B and the direction BC in which the line is to be continued.

For this purpose, we could chain up to A, set out AE (any length) at 90° to the chain line DA by optical square or construction, then set out EF at 90° to AE, to clear the obstacle, then FB at 90° to EF and equal in length to AE, and finally BC at 90° to

FB, to continue the chaining, the length AB being found by measuring EF.

This simple method, however, would be, practically, inaccurate, and we use an extension of the same principle as follows: Choose D and A, so that DA is at least as long as AB if possible. Set up perpendiculars DG and AE, equal in length. Then measure GE, which should check with DA, and measure the diagonals GA and DE, which should agree in length. Then range F and H in line with GE, set up perpendiculars BF and CH, each equal to DG or AE, and measure BC, BH, and FC to check, as before. Then BC will give the direction of the chain line, and AB is found by measuring EF.

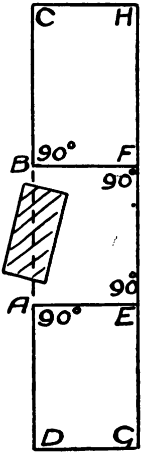


Fig. 32.

Much depends upon the accurate ranging of the line GEFH. It need hardly be said that such obstacles should be avoided if possible. But it sometimes happens that a haystack is built in a line after the stations have been arranged, but before that line is measured, and in some cases the ranging of a line past a house may much simplify the arrangement of lines.

For instance, referring to fig. 1, p. 1, suppose there was a house near C as shown by the dotted lines. It would, perhaps, be impossible to find a position for C, such that BC would be sufficiently near the road, and would clear both houses, while AC would also clear the buildings near L. In that case we must either measure some otherwise useless lines to fix the stations, or we may still arrange the lines as in the figure, and clear the second house on BC as above. In this case C would not be fixed in the first instance, but we would fix B, lay out BC_1 as far as C_1. Then clear the obstacle, and carry on the line to fix C in a suitable position for measuring CA. Such a line should only be used, intentionally, when the obstacle is near the end (in which case it should be dealt with as above), or when the direction of the line can be checked by looking *over* the obstacle from high ground on either side. Otherwise, if a good result is desired, it is a good plan to repeat the construction on both sides of the line.

Ranging Long Lines.—If the line to be measured is longer than about ten or twelve chains, intermediate points on it should be ranged before chaining begins.

To do this the surveyor stands a few yards behind the signal at one end, and directs an assistant with a ranging rod, right or left, until the rod comes into line between the two end signals. If AB are the two end stations it sometimes happens that this cannot be done directly, either because the line is too long for the signal at one end to be distinctly visible from the other end, or because there is rising ground in between, thus obstructing the view. In such cases the line can be ranged as follows: the surveyor stations himself at C, as near as possible to A, but so that he can just see the rod at B distinctly. An assistant goes to a point D, so that he can just see A, and holds out a ranging rod at arm's length. C then directs D, right or left, till he comes into line with

B, as shown (fig. 33). The assistant at D then steps behind his ranging rod, and directs the surveyor until a ranging rod carried by the latter comes into line with A, as shown at C_1. The surveyor then directs the assistant into line again, as shown at D_1, and so on till both are in line. It is a good plan to move a little *too far* each time, after the first, as the process is thereby expedited.

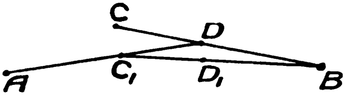


FIG. 33.

The Line Ranger.—The line ranger is an instrument whereby this can be done by one person. It consists of two right-angled isosceles glass prisms, one above the other, as shown in plan in fig. 34. One face of each prism lies in the common plane KL, but the planes of the longest faces are perpendicular, viz. KM and LN. A ray of light coming from B, and entering the lower prism, will undergo total reflection from the hypotenuse, and will enter the eye of the observer at D, as if coming from the direction CD. A ray from A to the upper prism will be similarly reflected from the hypotenuse NL, and if the instrument is exactly in the straight line AB, the

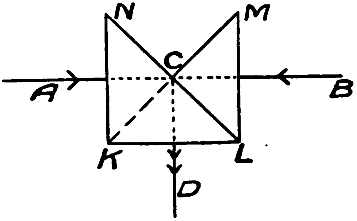


FIG. 34.

ranging rods at A and B will appear vertically above one another in the upper and lower prisms. To use the instrument, the surveyor guesses a point E (fig. 35) where he will be nearly in line, holds the line ranger to his eye so that he is looking into the common plane KL (fig. 34), and then walks about at right angles to AB till the rods at A and B appear in the same vertical line. The point F (fig. 35) is thus fixed.

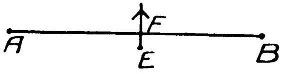


FIG. 35.

Adjustment of Line Ranger.—To test the adjustment, range out, by eye, four points A, B, C, D (fig. 36) in line. Then walk across with the instrument till a point E is found, where C and D coincide, as seen in the lower prism.

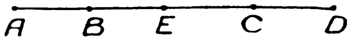


FIG. 36.

Then A and B, as seen in the upper prism, should coincide with one another and should be in the same vertical line as C and D.

If not, it will be found that one of the prisms is fitted at the back with adjusting screws, and these should be worked until the adjustment is perfect.

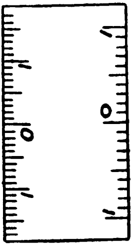


FIG. 37.

Plotting a Chain Survey.—For striking the arcs by which the stations are fixed on the plan, the beam-compass is the most convenient instrument, if the sides are at all long. A cheap and very useful form is that known as "Philpot's patent," made and sold by Messrs. Harling, Finsbury Pavement, London.

For plotting the offsets, an offset scale is employed. This consists of a short length of scale, divided and numbered to correspond with the main scale. Most conveniently the zero mark is in the middle (fig. 37), and the graduations go each way from that point.

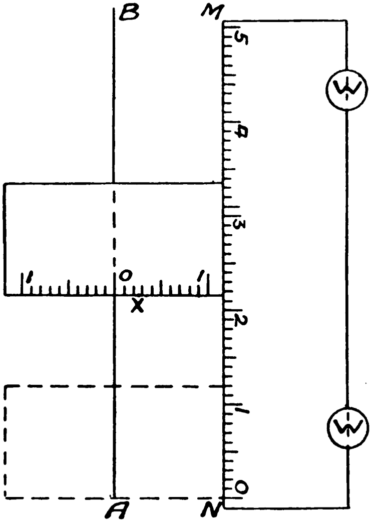


Fig. 38.

In plotting the offsets, say on a line AB (fig. 38), a line NM may be ruled parallel to AB, and at a distance from it equal to half the length of the offset scale. The offset scale is then placed with its centre point, or zero mark, on the station A, and its edge along the perpendicular AN, as shown by the broken lines. The main scale is then placed along NM, with its zero mark at N, coincident with the corner of the offset scale. The main scale is carefully fixed in that position by weights W.

For plotting any offset, the offset scale is then slid along till its edge comes to the proper distance along the main scale. The zero of the offset scale will remain on the line AB, and the proper length of the offset can then be pricked off with a fine needle-point. In the figure, the scale is arranged for plotting an offset at distance 2·16 chains. A mark at the point X would correspond with an offset of 26 links to the right.

In this description it has been assumed that the chainage starts from the station A. Otherwise, the edge AN, in the initial position of the offset scale, must, of course, pass through the zero of the chainage, and not through the station. Similarly, if there is an index mark such as D (fig. 39) on the offset scale, we use this mark for reading distances, instead of the edge of the scale. In this case, we place the offset scale so that its edge, AN (fig. 38), passes through the zero of the chainage, and then slide the main scale along NM until its zero mark coincides with the index D, and weight it down in that position. If the offset scale is graduated from one end to the other (fig. 39) we may still use it as above described, or we may place the edge of the main scale *along* the line AB, plot all offsets on one side of the line, then reverse the main scale, putting it on the other side of AB, for the remaining offsets.

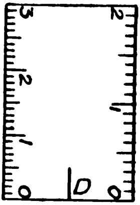


Fig. 39.

The student should accustom himself to read the figures with the field book upside down, or in any position, and when he is plotting any line, he should turn the field book so that the central column lies before him in a direction about parallel to that of the line on the paper, so that right and left in the field book will correspond in direction with right and left on the paper.

The sketch is best filled in, as a rule, after every two or three points are plotted.

The main scale is usually 12 inches long, so that with a scale of 1 chain to 1 inch, for instance, we can plot distances of 12 chains without moving the main scale. But if the line is longer than 12 chains, we should use only 10 chains length, and then move the scale forward. Thus an offset at 15·3 chains would be plotted at 5·3 on the second length. If we had used the whole length of 12 chains, before moving the scale, this offset would have been plotted at 3·3 on the second length, which is confusing.

Similarly with a scale of 2 chains to 1 inch, we would advance the main scale after every 20 chains.

Determination of Areas.—The areas of the various enclosures can be obtained by any one of several methods, some of which will now be described.

The Planimeter.—There are many different forms of this instrument on the market, of which the best known, and probably the most accurate, is the Amsler planimeter. Full instructions for use are supplied with the instrument, if required, and it is proposed here to enter only into a few precautions which should be observed for an accurate result.

The bar of the instrument is graduated, so that areas can be obtained in square centimetres, square inches, or other square units, according to the scale of the drawing. Now this graduation should not be trusted. The best plan is to rule out very carefully two independent rectangles of known dimensions, and preferably of the same area, and to planimeter them with any setting of the instrument. The results should agree within one part in 1000. Then planimeter the unknown area twice, with the same setting. These results being found also to agree, we thus establish a ratio between the unknown area and the known. Thus suppose the rectangles are (*a*) 8″ × 3″, (*b*) 6″ × 4″, the area being 24 sq. inches in each case. Suppose the planimeter readings are 17·56 and 17·55, mean = 17·555.

Now suppose for the unknown area we obtain 19·32 and 19·33, mean 19·325.

Then we have the proportion :—

Unknown area : 24 sq. inches : : 19·325 : 17·555.

$$\therefore \text{area} = \frac{24 \times 19{\cdot}325}{17{\cdot}555} = 26{\cdot}363 \text{ sq. inches.}$$

The rectangles should have their dimensions marked out with fine needle-points, directly from the scale, and, after they are drawn, each side should be carefully checked.

The Reduction to Acres.—When the area has been found in square inches, the result is reduced to square chains according to scale. If the scale be n chains to 1 inch, multiply by n^2. Thus if the scale of the above drawing were 3 chains to 1 inch, the area would be

$$26{\cdot}363 \times 3 \times 3 = 237{\cdot}267 \text{ sq. chains.}$$

Similarly for 20 links to 1 inch, it would be

$$26{\cdot}363 \times 0{\cdot}2 \times 0{\cdot}2 = 26{\cdot}363 \times {\cdot}04 = 1{\cdot}05452 \text{ sq. chains.}$$

To reduce square chains (Gunter's) to acres, divide by 10. Then multiply the decimal part by 4 to reduce to roods, and again the decimal part by 40 to reduce to perches.

$$\begin{array}{r} \text{Thus } 237{\cdot}267 \text{ sq. chains} \\ = 23{\cdot}7267 \text{ acres} \\ 4 \\ \hline 2{\cdot}9068 \text{ r} \\ 40 \\ \hline 36{\cdot}2720 \text{ s} \end{array}$$

or 23a. 2r. 36·3p. nearly.

Equalization of Boundaries.—In the absence of a planimeter, the writer has found this the most accurate method, if working from the plan. In fig. 40, the boundary of the area to be determined is shown by the broken line.

Taking a piece of transparent gelatine with a straight line scratched on one side (a protractor answers very well) we move this about, until the line XY on the gelatine very nearly coincides with a portion *ab* of the curved boundary, cutting off as much area on one side as it adds on on the other. Then mark the points XY with needle dots, remove the protractor and rule in the line *ab*. Repeat this all round the figure as shown, and we replace the curved boundary by a set of straight lines, which, if the work is properly done, will follow it so closely as to be almost indistinguishable from it at a glance, but which will everywhere cut off as much at one place as they add on at another.

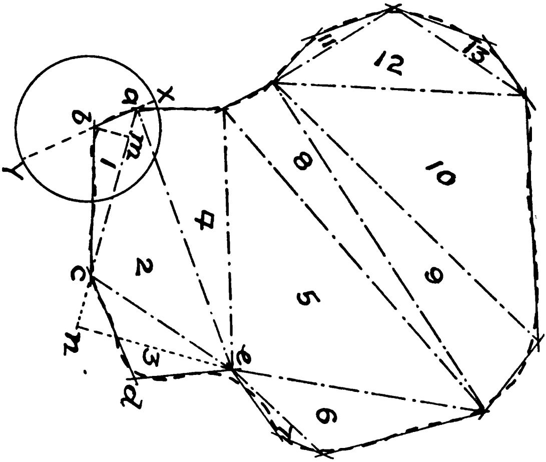


FIG. 40.

This equalization can be judged with a high degree of accuracy by eye, if the parts equalized are kept very small. All the lines should be ruled a little too long, as shown, so as to *cut* one another. We then go round the figure and mark the points of intersection, *a*, *b*, *c*, *d*, etc., with needle dots. Then divide the figure up into triangles by joining these dots in whatever order may seem most convenient, so as to obtain well-shaped triangles, and number as shown.

We then take the triangles in pairs having a common base, and for each pair we measure the common base, and also the perpendicular heights. Thus for the triangles numbers 1 and 2, the common base is *ac*, and the heights are *bm* and *en*. Then the areas of those triangles will be $\frac{1}{2}$ *ac* (*bm* + *en*).

Tabulate as follows:—

No. of Triangle.	Heights.	Sum.	Base.	Product.
1 and 2	$\left.\begin{matrix} bm \\ en \end{matrix}\right\}$	$(bm + en)$	*ac*	*ac* × sum

When the table is complete the last column is summed, and the result divided by two to find the area.

In some cases, as, for instance, the triangle No. 3 in the figure, it is more convenient to take one triangle at a time, in which case, of course, the product will be simply base × *ht*.

It is best to measure directly in feet or chains to scale, instead of in inches. The area is thus obtained directly in square feet or square chains.

Reducing Geometrically.—Instead of dividing into triangles as above, which is more or less tedious, the figure may be reduced to a single triangle geometrically (after the equalization of boundaries) as follows:—

Suppose the points 1, 2, etc. in fig. 41 correspond with *a*, *b*, etc. in fig. 40. Choose any one side as base, the only condition being that it should not, when produced, cut *through* the figure. Thus 1, 2, or 2, 3, would do, but we must not choose a line like 3, 4. Say we choose the side 1, 2. Then beginning at one end, 2, we join this point 2 to the *next point but one*, in this case 4, and from the intermediate point, 3, draw 3*f* parallel to 2, 4 to meet the base at *f*. Then the triangle 2, 3, 4, may be replaced by 2*f*4, which is equal in area. *f* is now the end of the base; join *f* to the next point but one in order, viz. 5, and from 4 draw a parallel to meet the base in *g*. Then join *g* to 6, and from 5 draw a parallel to meet the base in *h*. Join *h* to 7, and draw a parallel from 6, meeting the base in *i*, and join *i* to 8 and draw the parallel $7j$. The point 7 has thus been replaced by j, and having now arrived at the top of the figure, we rule in the line $j8$ and stop there.

Then begin at the other end, 1, of the base, and proceed similarly with the points 15, 14, etc. In this way we finally arrive at the figure $kj8$, which is equal in area to the original,

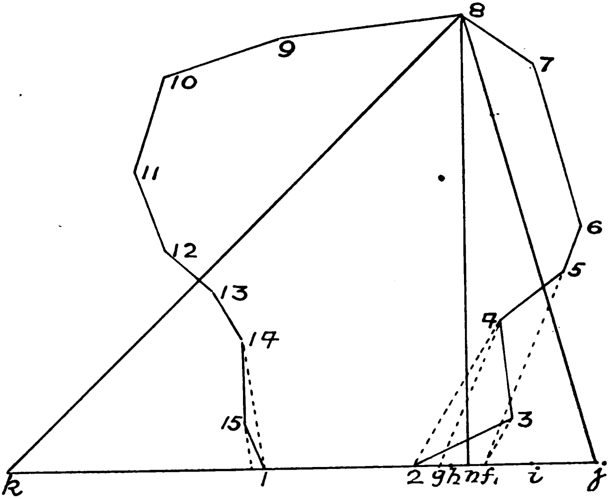


FIG. 41.

and the area of which is found by measuring the base kj, and the perpendicular height, $8n$.

Then area $= \frac{1}{2}\, kj \times 8n$.

In the opinion of the writer this method is less accurate than that first given, but it is also less tedious, and it gives a fair result if great care is taken with the parallels, and if all the points are marked with fine needle dots. It is not necessary to actually draw the lines such as 2, 4. We simply place the set square so that its edge passes truly through the centres of the dots at 2 and 4, then slide it along to draw the parallel $3f$.

The Computation Scale.—The computation scale, fig. 42, consists of a scale graduated to the scale of the drawing, and fitted with a transparent slider, as shown, on which a line XY is engraved. To use it, the area is divided into strips, *each* 1 *chain*

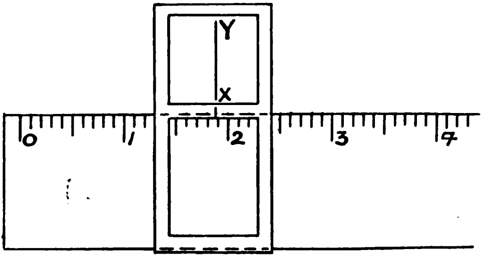


FIG. 42.

high to scale, as shown (fig. 43). The slider is set to zero on the main scale by means of the index mark shown below X, and the whole scale is then moved until the line XY occupies the position *ab*, where, as judged by eye, it cuts off, and adds on,

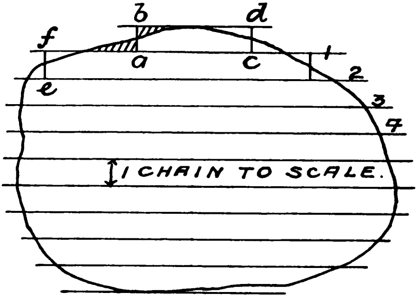


FIG. 43.

equal areas, which are shown shaded in the figure. The main scale is then held fast, and the slider moved until the line XY occupies the position *cd*, where it equalizes the areas at the other end of the first strip. Then obviously

Area of first strip = *bd* × *dc*. But as *dc* has been made 1 chain high, the area in square chains is given by the length *bd* in chains, which we can read off at once on the main scale.

Instead of reading it at once, it is usual to move the whole scale (the position of the slider on it remaining fixed) till the line XY occupies the position *ef*, where it equalizes the left end of the second strip. Then hold the scale, and move the slider, to equalize the right-hand end. The reading on the main scale will then give the sum of the areas of the two strips, and so on, until there is not sufficient length of scale left for another strip. Then note the reading, set the slider back to zero, and proceed with the remaining strips. The sum of all readings taken in this way will give the required area.

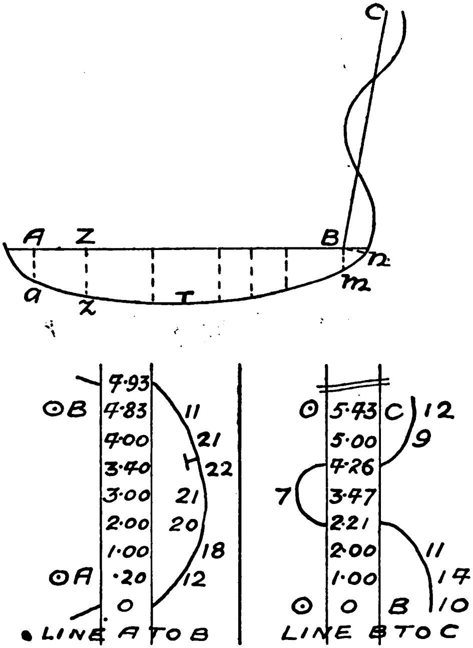


FIG. 44.

The last strip will seldom be just 1 chain in height. If not, we must deal separately with this strip. Obtain the length of the strip after equalization at both ends as above, and then multiply by the height of the strip in chains.

This method is less accurate than those already described, but it is quicker than the equalization of boundaries, and may be used in the absence of a planimeter. If obliged to work with a main scale not divided to the proper scale, we reduce the final result according to the ratio of the scales. Thus with a drawing done to a scale of 3 chains to 1 inch, and a computation scale divided 2 chains to 1 inch, we rule the lines *one-third* of an inch apart, and measure the accumulated length of strips in chains with the scale used, and then multiply by $\frac{3}{2}$. The proper scale of the drawing is to be taken as the numerator.

If the scale is large—say 20 links to 1 inch, for instance—one chain, to scale, would be an inconveniently great height for the strips. In such cases use some fraction of a chain—say one-tenth of a chain—and then take this same fraction of the final result.

Area from Offsets.—Suppose AB, BC (fig. 44) are two lines, and that their field books are as shown on page 54.

The positions of these offsets (which are not drawn to scale) along AB are shown by the dotted lines, and if we regard the portion of boundary between two adjacent offsets as straight, it is evident that the ground between the survey line and the boundary will be divided into a number of trapezia, the area of each of which can be found by multiplying the mean length of adjacent offsets by the distance between them. Thus area of

$$\mathrm{A}az\mathrm{Z} = \mathrm{AZ} \times \left(\frac{\mathrm{Z}z + \mathrm{A}a}{2}\right).$$

The calculation is shown in the table on next page, in which the offsets are taken from the field book above.

The field book shows that an allowance for "hedge and ditch" (see p. 31) is to be *subtracted* along the line AB. The area of this is reckoned for a width of 6 links over the whole length of line AB, viz. 493 links.

The corners are, perhaps, best dealt with separately as shown. If Bm, Bn (fig. 44) be the last and first offsets on AB and BC respectively, then if AB and BC are nearly at right angles, the area of the corner Bmn is very nearly given by Bm × Bn, which is the rule used in correcting for the corner area in the table.

But if the angle ABC is widely different from 90° (say less than 70° or more than 110°) we must mark the points m and n, measure mn, and calculate the area of the corner triangle from the known lengths of the three sides.

Line.	Offsets (Links).	Sum.	Distance (Links).	Product.	
				+	−
AB	12 18	30	80	2400	
	18 20	38	100	3800	
	20 21	41	100	4100	
	21 22	43	40	1720	
	22 21	43	60	2580	
	21 11	32	83	2656	
			463		
BC	10 14	24	100	2400	
	14 11	25	100	2500	
	11 0	11	21	231	
	0 7 −	− 7	126		882
	7 0 −	− 7	79		553
	0 9	9	74	666	
	9 12	21	43	903	
			543		
				23,956 1435	1435
				22,521	

allowance for ditch

$= 493 \times 6 =$ − 2958 $\quad 2)\ 22{,}521$

corner $\frac{11 \times 10}{2} = 55$ } 175 $\quad$ 11,260 sq. links

corner $\frac{20 \times 12}{2} = 120$ } − 2783 $\quad$ 2,783

8,477 sq. links

0·8477 sq. chains

The distances in the table are got by repeated subtraction, that is, each distance in the field book—beginning from A—is subtracted from the next above it. Before working out the products, the distance column is summed to see that it checks with the length of the line—from station to station—as given in the field book.

When the boundary crosses, and comes *within* the survey lines, as shown along BC, these parts of the area are to be deducted, and are marked with a *minus* sign in the table for this reason.

It is most convenient to work with the *sum* of the adjacent offsets, as in this table, and to *halve* (once for all) the result obtained by summing the product column, as shown.

This rule, whereby each small area is regarded as a trapezium, and its area found in that assumption, is called the *trapezoidal rule.*

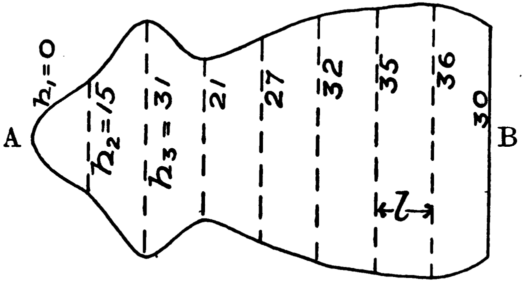


FIG. 45.

The area enclosed within the survey lines themselves may be found by plotting the figure and reducing to a triangle, or by dividing it into a number of triangles, and working as described under *equalization of boundaries.*

If this method of finding the area is to be used at all, it is most usual to divide the polygon formed by the survey lines into triangles *on the field,* to lay out the perpendicular height of each triangle with the optical square, and to measure it *on the ground,* so that no measurements at all have to be taken from the drawing.

Simpson's Rule.—Simpson's rule is particularly useful in finding the areas of long strips of ground, where the breadths are known at equal intervals.

If AB (fig. 45) be the whole length, it must be divided into any *even* number of equal parts, and the breadths, h_1, h_2, etc., must be known at the ends of each part. Thus there will be an *odd* number of *ordinates* or breadths. Let the length of one part be l. Then, by Simpson's rule,

Area $= \frac{l}{3}$ (first and last ordinates + 4 × sum of *even* ordinates + 2 × sum of *odd* ordinates, not including the first and last).

The even ordinates are those numbered with an *even* number in the figure, viz. h_2, h_4, etc.

The odd ordinates are numbered with odd numbers.

That is, area $= \frac{l}{3}(h_1 + 4h_2 + 2h_3 + 4h_4 + \ldots + h_9)$.

The calculations should be tabulated as in the example below, in which the ordinates are supposed to have the values marked on the figure, and l is supposed to be 1 chain or 66 feet.

No.	Ordinate.	Ditto × Factor.
1	0	0
2	15	60
3	31	62
4	21	84
5	27	54
6	32	128
7	35	70
8	36	144
9	30	30
		632

For the first and last ordinates, the multiplying factor is unity. For the others, either four or two, according to the number. The last column is summed, and multiplied by $\frac{l}{3}$.

In this case area = 22 × 632 = 13882 sq. feet.

Unless the breadths are known at regular intervals it is better to use the trapezoidal rule.

Simpson's rule may also be used to find the *volumes* of embankments, etc., by taking as ordinates the *areas* of the cross sections at regular intervals along the centre line.

Area of a Triangle.—When the three sides of a triangle are known, the area may be found by formula. Let a, b, c be the lengths of the three sides, and s the half-sum, so that $s = (a + b + c) \div 2$.

Then area $= \sqrt{s \,.\, (s - a) \,.\, (s - b) \,.\, (s - c)}$.

Degree of Accuracy in Chaining.—It is usually considered, as the result of tests, that on good ground with careful work, the error in chaining a length by the methods described in this chapter should not exceed one part in 1000. On rougher ground one part in 250 is a fair average.

North Point.—It is usually desirable to show the north point on the plan for the sake of finish, even if it is otherwise of no importance. Approximate methods of finding it are:—

(*a*) By means of the magnetic compass. This will give the direction of magnetic North—true or geographical North can then be shown by taking the magnetic declination from the Admiralty Chart. Unless *true* North is shown in this way, it should be stated in the plan that magnetic North has been shown, and the date should be named.

(*b*) If local time is fairly accurately known, by observing the direction of the sun at 12 o'clock noon. In the northern hemisphere this will give the direction of the *South* point. If a very accurate result is required, the "equation of time" should be obtained for the day from the Nautical Almanac. Suppose it is given as 15 m. 10 sec. to be subtracted from mean time. Then we must add this to 12 o'clock, and take the position of the sun at 12 h. 15 m. 10 s.

(*c*) Without an accurate knowledge of local time, by setting up a vertical stick AB, on level ground. Then if AC be the shadow cast by this stick about three hours before noon, we mark the end C of the shadow, and then with a piece of string we mark out the circle CED on the ground, with A as centre and AC as radius. As the sun rises and sets the end of the shadow will describe a path such as CFD, and we watch it, about three hours after noon, until the end of the shadow returns to the circle CED, at the point D. This point is then also marked, and if CD be bisected and the point so found be joined to A, this line gives the direction of the meridian. The stick must be carefully plumbed, but even with great care the method is not susceptible of great accuracy.

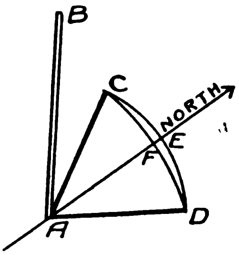


FIG. 46.

EXERCISES FOR REVISION.

1. State the relative advantages and disadvantages of :—

(*a*) the band and ordinary chains ;

(*b*) the Gunter's and 100-foot chains, saying when you would use each.

2. If the possible error in laying out the direction of an offset is 5°, and the scale of the drawing is to be 50 feet to 1 inch, how long may an offset be, in order that the displacement of a point of the paper may not exceed ·01 inch ?

3. If an offset can be measured with an accuracy of one part in 150, and if the error in laying out the direction may be 4°, prove that the displacement of a point on the paper, due to both sources of error, may be $\frac{l}{n}\sqrt{\sin^2 4^\circ + (\frac{1}{150})^2}$ inches, where l is the length of offset in links (or feet) and n the number of

links (or feet) to the inch in the scale. Hence find l, if the scale be 50 feet to 1 inch, so that the displacement may not exceed ·01 inch.

4. Enumerate the conditions to be borne in mind when arranging the lines for a chain survey.

5. In a certain survey with hedge and ditch fences, the lines were all *inside* the area. The hedge was next the line on three lines whose lengths were 7·03, 5·61, and 4·79 chains, and the ditch was next the line on one line, whose length was 6·83 chains. Find the allowance in area for hedge and ditch, stating whether it is to be added or subtracted.

6. Find, by Simpson's rule, the area of the pieces of ground whose breadths are given in the tables below :—

(*a*) Distance in chains	0	·5	1·0	1·5	2·0	2·5	3·0	3·5	4·0
Breadth in links	18	21	22	17	12	13	20	18	18

Distance in chains	0	1·0	2·0	3·0	4·0	4·5	5·0	5·25	5·5	5·75	6·0
Breadth in links	0	10	17	20	15	22	17	19	15	19	16

7. For what purpose, and how, is a clinometer used in chain surveying? Explain how the results of the clinometer measurements are used.

8. How is the adjustment of the clinometer tested?

9. What is the use of an optical square, and how is its adjustment tested?

10. Sketch and describe the line ranger, and say how you would test its adjustment.

11. It is desired to chain a line about 12 chains long, but in consequence of a mist a ranging rod cannot be seen farther than about 7½ chains. How would you range out the line (*a*) with, (*b*) without using, a line ranger?

12. Plot the notes on pp. 6 to 8.

CHAPTER II.

TRAVERSE SURVEYING.

General Principles.—A traverse survey differs from a simple chain survey mainly in the fact that the relative directions of the lines are fixed by means of angular measurements, and not from the lengths alone.

A traverse is usually applied to larger surveys, and particularly when the survey is along a definite *line* or *route*, such as a road or railway or coast line.

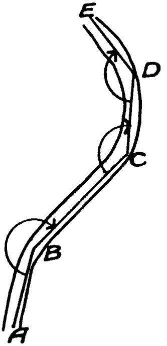


FIG. 47.

Let ABCDE be an arrangement of straight lines approximately following, say, an existing road. We chain along these lines, and take offsets to the sides of the road, just as in chain surveying.

But to fix the directions of the lines with respect to one another, we either measure the included angles between the lines with a theodolite or sextant, or, what comes to the same thing, measure the "bearing" of each line,—that is, the angle it makes with some standard direction, usually a north and south line.

We will suppose first that the included angles are being measured with a theodolite.

The Theodolite.—The most commonly used form of this instrument in England is now the transit theodolite, which is shown in fig. 48. Like all other forms, it consists essentially of a horizontal circle

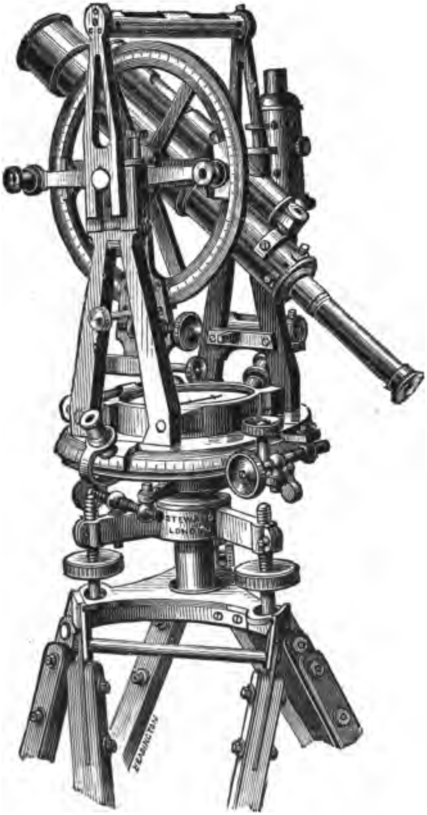


Fig. 48.

graduated in degrees and minutes, and of a second horizontal circle, concentric with the first but rotating independently of it, and carrying a telescope and a vernier or index of some sort, by means of which we can read, on the lower circle, the angle through which the telescope has turned in a horizontal direction. These circles are called the *lower* and *upper* plate or limb respectively. Each is provided with a *clamping screw* to fix it in any desired position, and a tangent screw to give it a slow motion after the clamp is fixed.

The whole can be "levelled"—that is, the horizontal circles can be brought into a truly horizontal plane—by an arrangement of three or four *plate screws* underneath and one or more spirit levels on the plates, and the telescope is capable of motion in a vertical plane, so as to enable stations to be viewed which are either higher or lower than the observer's station.

A plumb-bob can be hung either from the tripod or from the instrument itself, so as to enable it to be centred accurately over a station.

It will be assumed that the reader has studied an actual theodolite and learned the names of the parts on the instrument itself.

To use it for measuring say the angle ABC between two traverse lines, remove the ranging rod (if any) at B and then—

(1) *Centre* the instrument over the station by means of the plumb-bob. *Practice makes perfect* in this operation, but if the beginner lifts *two* legs of the tripod off the ground, he will find it possible to move the instrument in any direction, whilst still keeping the horizontal circles level, as well as can be judged by eye. When nearly right the raised legs are pressed down one at a time, without shifting the position of the instrument. The legs are then pressed into the ground to make all firm, and

the centring can be improved by pressing some more firmly than others. Some instruments have centring plates, by means of which the final adjustment can be very accurately made.

(2) *Level* the horizontal plate. If there are *three* levelling screws, set the spirit level parallel to *two* of them, and work these, either both thumbs in or both thumbs out, until the bubble is central. Then turn the plate through 90°, so that the level is at right angles to its first position, and level again, using the *third screw only*. Then repeat. (If there are two levels at right angles, do not turn the plate, but use the second level instead when adjusting with the single screw.)

If there are *four* screws, set the bubble parallel to two *opposite* screws. Look to see at which end of its run the bubble is situated, and *slacken the screw under that end*, about one turn. See whether in doing this we work "thumb in," or "thumb out," and accordingly work the two opposite screws, either both thumbs in, or both thumbs out, *until the bubble goes a little too far*. Then tighten up the screw which was first slackened, so as to bring the bubble back to the centre; repeat this with the other pair of opposite screws, and then again with the first. It will be observed that with a four-screw instrument, the upper portions are usually held down by a ball and socket joint, so that we must begin by *slackening* one screw. Working in the above manner, we ensure that all the screws are tight when the levelling is finished.

(3) Look through the telescope at the sky, or at a white paper held just in front, and work the eyepiece in or out slowly, so as to bring the "cross hairs" or spider's webs (which will be seen inside the telescope) sharply into focus. Turn the eyepiece *clockwise*, whilst pushing it in or out, to avoid unscrewing the lens.

This should not need adjustment after the first station.

(4) Set the vernier on the horizontal circle to zero, or 360°, using the tangent screw for the final adjustment, after clamping.

(5) Unclamp the lower plate, and aim the telescope at the *back station* next behind (in this case supposed to be A) taking aim *over the top* of the telescope only. Clamp the lower plate.

(6) Look through the telescope, set the elevation about right, and focus the object by the focussing screw at the side. If the aim was properly taken, it will be possible to *see* the signal at A in the field, though it will not be exactly on the hairs. Bring the intersection of the hairs near the lowest visible point of the signal, and clamp the vertical motion. Then use the lower and vertical tangent screws, so that the centre of the spider's webs exactly bisects the lowest visible point of the signal. Move the eye about, and if the hairs appear to move on the signal, touch the focussing screw slightly, till all movement of the hair ceases. Then check the vernier to see that it still reads zero.

(7) Unclamp the *top plate*, take aim over the top at the *forward* station (in this case C) and repeat the operations described in (6), except that we use the *upper tangent screw* this time, and, of course, the vernier will no longer read zero, but will give, instead, the included angle ABC, *measured clockwise from the back station.*

(8) For good work the angle should be checked by the method of *repetition.* After it has been read and booked, unclamp the *lower* plate again, and direct again to the back station, as before, using the lower tangent screw, and checking the vernier at the end to see that it still reads the angle already booked. Then unclamp the upper plate, and direct again to the forward station as in (7), when the vernier should give a reading *double that which has been already booked.* This check is duly entered in the field book. If the angle does not double correctly, it must be measured a third time.

Great care must be taken always to read the same vernier, if only one is read.

Plotting the Survey.—It will be evident that when the lengths of the lines are known, and also the angles between them, they can be drawn on paper by means of an ordinary scale and protractor. But this method of plotting is objectionable, because the direction of each line is obtained from that of the one before, and the unavoidable errors of drawing accumulate, in consequence, from line to line.

To avoid this, it is usual to divide the paper up into squares, each containing some round number of units according to scale. A point A (fig. 49) is taken as origin, the bottom and left-hand lines AB, AC, are taken as axes and *co-ordinates* are calculated for each point. Thus if P be any point of the traverse, PQ and AQ are its co-ordinates, and when they are known the point P can be plotted independently of all others.

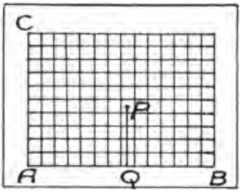


FIG. 49.

In English practice PQ and AQ are usually called the *north* and *east* co-ordinates respectively, but in the Colonies they are often called the *y* and *x* co-ordinates.

Calculation of Bearings.—The bearing of a line in English practice is generally understood to mean the angle which that line makes with some standard direction, this angle being measured *clockwise from the standard direction* right round the circle. Thus if AN be the standard direction (fig. 50), the bearing of AB is 120°, and that of AC is 300°.

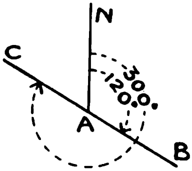


FIG. 50.

The standard direction is nearly always taken as vertical on the paper, and may be a true north or south line, or it may be any assumed direction, chosen either so as to make the survey fit well on the paper, or for any other reason.

In either case we must obtain, to start with, the bearing of the first line of the traverse. This may be found as hereafter described if a true north and south line is taken, or it is *assumed* at the desired figure if the standard direction is simply being chosen. In calculating the co-ordinates, then, the first step is to calculate the bearings of all the other lines from the included angles as follows.

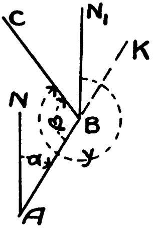


Fig. 51

If AN (fig. 51) be the standard direction, AB the first line, put α = bearing of first line.

β = included angle between the lines, measured as already described.

These angles are marked on the figure, and the problem is, given α and β to find the bearing of the next line BC.

Draw BN_1 parallel to AN. Then BN_1 is the standard direction at B, and the required bearing is the clockwise angle N_1BC, marked γ on the figure.

Produce AB to K. Then $N_1BK = \alpha$.

$$\text{and } KBA = 180^\circ.$$

$$\text{and } ABC = \beta.$$

$$\text{But } \gamma = N_1BK + KBA + ABC$$

$$\therefore \underline{\gamma = \alpha + \beta + 180^\circ.}$$

This rule is always true, but in some cases it will be found that, for certain values of α and β, the result is more than 360°. Now in this case the correct bearing is to be found by subtracting 360° from the above result. Thus if $\alpha + \beta + 180^\circ = 420^\circ$, it is obvious that this means one complete revolution of $360^\circ + 60$, and the latter alone is of importance. Hence in such cases, the bearing

$$= \alpha + \beta + 180^\circ - 360^\circ$$
$$= \underline{\alpha + \beta - 180^\circ}$$

Now it is evident also that this will be the case only when $\alpha + \beta$ is greater than 180°. Hence we have the rule :—

To find the bearing of any line, take the bearing of the previous line, and to it add the included angle between the lines. Then if the result is *less* than 180°, *add* 180° ; but if the result is *greater* than 180°, *subtract* 180°.

In exceptional cases the result, after subtracting 180°, will still be greater than 360°. In such cases, we subtract an additional 360°, for the same reason as before.

Examples.—Calculate the bearing of BC in each of the following cases : Bearing of AB = (*a*) 37° 21′ ; (*b*) 9° 184′ ; (*c*) 271° 29′. Included angle ABC = (*a*) 114° 31′ ; (*b*) 257° 31′ ; (*c*) 298° 47′.

(*a*)		(*b*)		(*c*)	
37° 21′		94° 18′		271° 29′	
114° 31′		257° 31′		298° 47′	
151° 52′		351° 49′		570° 16′	
180° 0′		180° 0′		180° 0′	
331° 52′	Ans.	171° 49′	Ans.	390° 16′	
				360° 0′	
				30° 16′	Ans.

In each case we add the included angle to the previous bearing. In (*a*) we add 180°, because the sum is less than 180°, and so on.

Whole-circle Bearing, and Azimuth.—The bearing of a line as here defined is sometimes called the "whole-circle" bearing, to distinguish it from the "reduced" bearing, to be hereafter defined.

In some American and Colonial practice, the bearing is understood to be measured from *south*, through west, or clockwise, and it is often called the "Azimuth" of the line. Then if the azimuth of the first line be obtained, the remaining azimuths are calculated as above, on the assumption that the standard direction (in this case the direction of due south) remains parallel to itself. But, in consequence of the roundness of the Earth, this is not actually the case (except at the equator), and hence, after travelling east or west for some distance, the calculated azimuth of any line will differ from the value we would obtain by direct measurement from due south at the point we had then reached.

In the best English practice the word azimuth is therefore used to mean the true bearing or azimuth of the line, as measured directly, at the beginning of the line, from the standard direction, which in our case is due north. The word bearing is then used for the bearing of any other line (or of the same line at any other point in it), such bearing being found from the included angles as already described on the assumption that the standard direction remains parallel to itself.

This practice seems sound and commendable.

Departures and Differences of Latitude.—Let AN (fig. 52) be the standard direction (assumed to be north for purposes of calculation) and AB any line. Draw BH perpendicular to AN. Then in going from A to B, it is clear that we go *North* by an amount AH, which is called the *difference of latitude* between A and B; and *East* by an amount HB, which is called the *departure*.

Now NAB, or a, is the bearing of AB and AH $=$ AB cos a; HB $=$ AB sin a.

Hence we have the rules:—

Difference of latitude between any two points $=$ *length of line* × *cosine of bearing*, and departure $=$ *length of line* × *sine of bearing.*

Reduced Bearings.—It is evident that if the line be in the direction CD (fig. 53) the difference of latitude is CH, and is *south* in direction.

Now the bearing of the line is the angle NCD, or a. But difference of latitude $=$ CH $=$ CD cos β, and departure $=$ HD $=$ CD sin β.

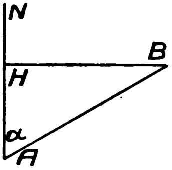


FIG. 52.

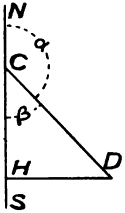


FIG. 53.

Hence to calculate in this case, we still use the rules given above, but we must use the angle β-instead of the true bearing.

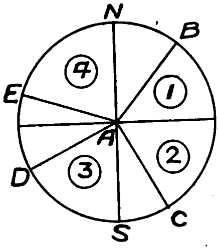


FIG. 54.

This is called the *reduced bearing* of the *line,* and, in order that the rules for difference of latitude and departure may remain always the same, it is to be understood that the reduced bearing, from which we calculate, must always be measured from the *north or south* line, whichever is the nearer, never from east or west.

Thus referring to fig. 54, if AN be the standard direction, a line may lie in any one of the four quadrants there numbered.

In quadrant number 1, the whole circle bearing lies between 0° and 90° (= NAB), and reduced bearing also = NAB.

Hence, calling the whole-circle bearing WCB, and the reduced bearing RB, we have :—

No. of Quadrant.	Limiting Values of W.C.B.	R.B. on Fig. 54.	Rule for R.B.	Cardinal Points.
1	0° to 90°	NAB	= W.C.B.	N.E.
2	90° to 180°	SAC	= 180° − W.C.B.	S.E.
3	180° to 270°	SAD	= W.C.B. − 180°	S.W.
4	270° to 360°	NAE	= 360° − W.C.B.	N.W.

The cardinal points are used to indicate the quadrant in which the line lies. Thus if W.C.B. = 163° 12′, the line lies in the second quadrant, and the cardinal points are S.E., while the reduced bearing = 180 − W.C.B. = 16° 48′. This angle would be used in calculating the difference of latitude and departure, and the former would be entered as *south* the latter being *east*.

Different Notations for Bearings.—In view of the fact, already pointed out, that neither the system of measurement nor the nomenclature relating to bearings, is universal at present, the surveyor should state in each case what system has been adopted. Thus if we write bearing of AB = 327° 12′ E. of N., we mean that starting from north we must go 327° 12′

through east, and the line will lie as shown in fig. 55. The same line would bear 32° 48′ W. of N., or, as it is sometimes written, N. 32° 48′ W.

Northings, Southings, etc.—A difference of latitude, if north is called a *northing*, and if south a *southing*, and departures may be similarly *eastings* or *westings*.

Checks on Closed Traverses.—A traverse which, starting from a point A, returns to that same point in a limited number of sides, is said to be *closed* (fig. 56).

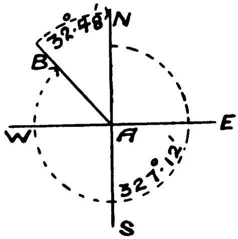


Fig. 55.

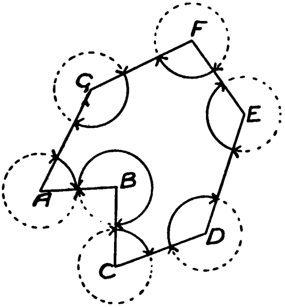


Fig. 56.

If we go right round such a traverse, we have the following checks :—

1. The sum of the observed included angles should be $2n \times 90° + 360°$ if *exterior* angles are measured, or $2n \times 90° - 360°$ if *interior* angles are measured, n being the number of sides in the traverse.

It will be remembered that the angles are measured clockwise from the back station. Hence we will measure interior angles (as shown by the full arrows) if we go round the traverse in the anticlockwise direction, ABC, etc. But if we go round in the clockwise direction AGF, etc., we will measure exterior angles as shown by the broken arrows.

The error in this summation should not exceed $\sqrt{n} \times e$, where e is the smallest reading that can be taken with the theodolite being used. Any small error, not much exceeding this amount, is distributed over the various angles, so as to make them sum correctly. Correct preferably those angles which are adjacent to the shortest sides, as fruitful sources of error are bad centring over the station and observing a sloping signal. Either of these will cause a bigger angular error over a short line than over a long one.

2. The sum of all the northings must be the same as the sum of the southings. Similarly the eastings and westings must sum up equal to one another. This is obvious, as, in completing the circuit, we have no resultant movement either in the north and south or in the east and west direction.

This checks the chainage, as well as the calculation of differences of latitude and departures. Hence the allowable amount of closing error depends upon the estimated accuracy of the chaining, which has been already referred to (p. 59). Thus if chaining to one part in 1000, the closing error in departure should not exceed the thousandth part of the total eastings and westings together. Any closing error within these limits is distributed over the whole, the correction to each being proportional to its value.

If we find that the southings add up to, say, 88 feet (or links) more than the northings, and the eastings to forty-two more than the westings, we conclude either that there is an error in the calculation or that a chain has been dropped on a line running *north-west* (as the northings and westings are too small) at such a bearing that its difference of latitude is about double its departure (in the ratio of 88 to 42) and we would look for such a line and re-measure it.

Traverse Sheet.—The annexed table shows the form in which the calculations are arranged, and which is called the traverse sheet. It refers to a five-sided closed traverse for which the angles and distances are shown in fig. 57 (not to scale). The student should work carefully through it for himself.

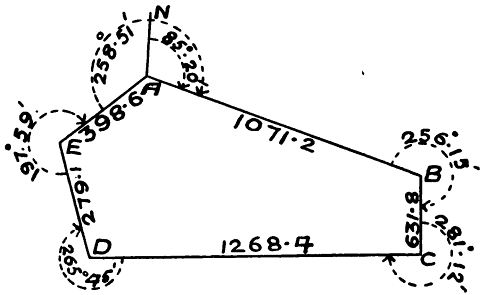


FIG. 57.

It will be seen that the closing error in the angles is two minutes, the sum being 1260° 2′. The angles adjacent to the shortest side, DE, are adjusted by one minute each, and the whole circle bearings calculated by the rules given, working from the given bearing of AB at the top, back again to the same figure at the bottom, as a check. The northings and southings sum with an error of two links, so that a correction of + 1·0 is distributed over the northings, and − 1·0 over the southings. Eastings and westings are similarly treated, and the co-ordinates of A

Point.	Observed Angle.	Corrected Angle.	Whole Circle Bearing.	Reduced Bearing.	Cardinal Points.	Distance (Links).	Difference of Latitude.				Departure.				Co-ordinates.		Point.
							N.	Corr.	S.	Corr.	E.	Corr.	W.	Corr.	N.	E.	
A															700·0	100·0	A
			85° 20′	85° 20′	N.E.	1071·2	87·2	+ ·1	—		1067·7	+ ·8	—				
B	256° 15′	256° 15′													787·3	1168·5	B
			161° 35′	18° 25′	S.E.	631·8	—		599·4	– ·8	199·6	+ ·2	—				
C	281° 12′	281° 12′													188·7	368·3	C
			262° 47′	82° 47′	S.W.	1268·4	—		159·4	– ·2	—		1258·7	– 1·0			
D	265° 45′	265° 44′													29·5	110·0	D
			348° 31′	11° 29′	N.W.	279·1	273·5	+ ·4	—		—		55·6				
E	197° 59′	197° 58′													303·4	55·0	E
			6° 29′	6° 29	N.E.	398·6	396·1	+ ·5	—		45·0		—				
A	258° 51′	258° 51′					756·8	1·0	758·8	1·0	1312·3	1·0	1314·3	1·0	700·0	100·0	A
			85° 20′				+1·0		–1·0		+1·0		–1·0				
B							757·8		757·8		1313·3		1313·3				B
	1260° 2′	1260° 0′															

being chosen, the rest are worked out as shown, and check back to A at the bottom.

Direct Reading of Bearings.—Thus to avoid accumulation of drawing errors, a traverse should never be plotted from included angles.

Even when it is desired to plot by scale and protractor, the whole circle bearing of each line should be worked out and used for plotting in preference to the included angles, the direction of each line on paper being thus obtained independently from the standard direction. Many surveyors prefer to obtain the bearings directly in the field, instead of measuring the included angles.

This can be done by compass, of course, but it can be done more accurately by specially manipulating the theodolite.

Let AB (fig. 58) be the first line, and suppose that its bearing, α, is known, or measured directly from north at A.

Then take the theodolite to B with the vernier clamped at this angle α. Unclamp the lower plate, direct back to A, clamping again. Then see that the vernier still reads α. Then unclamp the top plate, and turn the telescope to look at C. Then obviously the vernier will read $\alpha + \beta$. Now there are generally two verniers, 180° apart. Hence if we read the *opposite* vernier, this will automatically add or subtract 180°, and will give us the value of $\alpha + \beta \pm 180°$, which as already shown will be the bearing of BC. Then go to C, leaving this vernier clamped

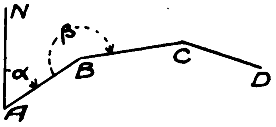


FIG. 58.

at this angle, direct back to B by unclamping the lower plate, see that the vernier reading is unaltered, unclamp the upper plate and direct to D, then read off the bearing of CD, again on the opposite vernier, that is, on the one first used.

Comparison of Methods.—The method last described saves some calculation, and, if bearings are taken from magnetic north, it affords a rough direct check by means of the compass attached to the instrument.

If the compass be of the old-fashioned circular type, reading from zero to 360°, we can read the bearing by it at each station, and check with the result from the vernier. But if it be of the more modern box compass type, attached to the lower plate and reading only through a small arc on each side of zero, we obtain a check at *alternate* stations only. Thus if we bring the needle to zero at A (fig. 58), at B the north end will be pointing south and we can take no reading. But at C it should come back to zero after we unclamp the lower plate and direct back to B.

The disadvantage of the method is that we cannot double the angles by repetition (p. 66) as a check. Then, if on directing back to B from C as already described, we check the reading of the vernier and find that it does *not* agree with the bearing of BC as booked at B, the discrepancy may be due to any of the following causes:—

1. An incorrect reading or incorrect booking at B.

2. A slip of the clamping screw while moving the instrument from B to C.

3. The use of the wrong tangent screw in directing back from C to B.

It is usual to assume that the error is due to one of the last two causes, and to reset the vernier to read the angle booked at B, and again direct on to B with the lower screw. But such an assumption is evidently unsafe, unless the angles at each station are read and checked with very great care.

Prismatic Compass Traverse.—The prismatic compass (fig. 59) consists of a small bar magnet, balanced on a pivot at its centre, and carrying a graduated ring or disc which turns with it. The zero of the ring is at the south end of the needle, and the graduations go thence clockwise (through west) round to 360°. The whole is carried in a circular box which can be aimed along the line whose bearing is required by means of two sighting vanes at opposite extremities of a diameter. The eye is then lowered slightly so as to look into the prism attached to the instrument. The graduations on the ring will then be seen magnified, by reflection, and we read the one which appears to be in line with the hair of the forward sighting vane.

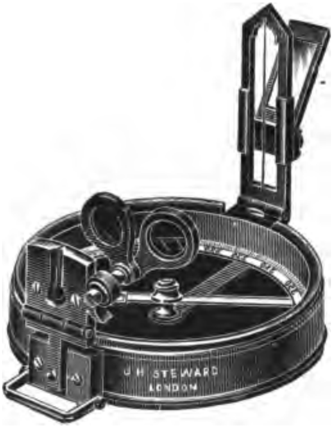


FIG. 59.

Thus when looking due north, the bearing should be zero. But the eye will be at the *south* end of the needle (as we are aiming north), hence the bearing will be read at the south end, where the eye is situated. This explains why the graduations start from zero at the south end.

Attached to the forward vane there is usually a small reflector of blackened glass which can be set to any angle and used for observing, by reflection, any object (either celestial or terrestrial) which is either too high or too low to be observed directly through the ordinary sights.

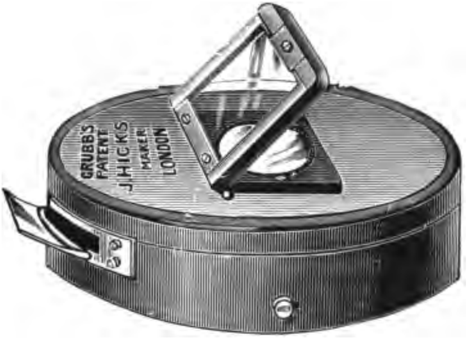


Fig. 60.

The prismatic reflector through which the readings are taken can be raised or lowered so as to focus the graduations distinctly, and on the opposite side is a small brake, which is pressed to check excessive swinging of the needle.

The Grubb Sight.—In fig. 60 is shown the instrument as made by Mr. J. Hicks of Hatton Garden, London, with the Grubb semi-transparent sight. With this arrangement we look into the instrument and see the graduated scale, as it were, superimposed on the object whose bearing is required, and we read the bearing without the use of any sighting hair or vane. Particulars can be obtained from the makers. The same form of sight is applied to the clinometer, optical square, hand level, and sextant.

Method of Use.—With the prismatic compass, all bearings should be read *forwards and backwards.*

The two values for each line should differ by 180°. If they do so nearly, the *back* bearing is increased or diminished by 180°, and the mean between this result and the forward bearing is taken.

This enables any local attraction of the needle at any station to be discovered. Thus suppose it be found that the back bearing of BC (in other words, the bearing of CB), after correction by 180°, is 3° greater than that of BC. Then, unless an error of reading or booking has been made, there is a local attraction of 3° at B *or* C. Now if the back bearing of AB agrees with its forward bearing, the error is not at B. If the back bearing of CD is 3° too small as compared with the forward, then there is local attraction at C. But if the back bearing of CD agrees with the forward, then there was probably a wrong reading.

Problems in Traversing.—Many problems arise in connexion with traverse surveys, which can be solved by the use of the following rules :—

(*a*) To find the back bearing of a line, given its forward bearing.

Rule : The back bearing = forward bearing ± 180°.

Example : If the bearing of AB be 125° 12′, what is the bearing of BA ? Answer: 125° 12′ + 180° = 305° 12′.

(*b*) To find the angle between two lines whose bearings are given.

Rule : Express *both* bearings as if measured *from* the point where the lines meet. Then the difference between them will give the clockwise angle *from* the line with the smaller bearing.

Example : In fig. 61 if the bearing of AB = 65° 12′, and that of CD = 112° 47′, find the angle DEA.

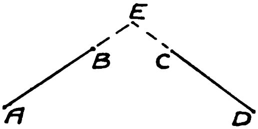


Fig. 61.

Bearing of AE = 65° 12′.
∴ ,, ,, EA = 245° 12′ } both measured
and ,, ,, ED = 112° 47′ } *from* E.
angle DEA = 132° 25′ clockwise.

(*c*) To find the bearing of a line given the co-ordinates of its ends.

Rule : Find the departure = difference between the *East* or x co-ordinates; and the diff. of lat. = difference between the north co-ordinates. Then tangent of reduced bearing

$$= \frac{\text{departure}}{\text{diff. of lat.}}$$

This gives the R.B. Then plot the points roughly (not to scale) on a diagram from the co-ordinates and see in which quadrant the line lies (see p. 71) and accordingly deduce the whole circle bearing.

Example : The co-ordinates of A and B are as follows :—

Point.	N.	E.
A	673·4	1219·4
B	771·8	276·3

Find the bearing of AB.

$$\text{Departure} = 1219{\cdot}4 - 276{\cdot}3 = 943{\cdot}1$$
$$\text{Diff. of Lat.} = 771{\cdot}8 - 673{\cdot}4 = 98{\cdot}4$$
$$\therefore \tan \text{R.B.} = \frac{943{\cdot}1}{98{\cdot}4} \therefore \text{R.B.} = 84^\circ\ 2'\ 36''.$$

And evidently B is N.W. of A, as its north co-ordinate is greater than that of A, but its east co-ordinate is less. Hence AB lies in the fourth quadrant, and evidently (fig. 62) the W.C.B. = 360° − R.B. = 275° 57′ 24″. If the bearing of BA was required, the cardinal points would have been S.E., and W.C.B. = 180° − R.B. = 95° 57′ 24″.

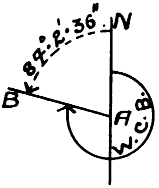


FIG. 62.

(*d*) To find the length of a line given the co-ordinates of its ends.

First Rule: Find the reduced bearing as above.

$$\text{Then length} = \text{departure} \times \text{cosec. R.B.}$$
$$\text{or length} = \text{diff. of lat.} \times \text{sec. R.B.}$$

Use the first or second of these according to whether the departure or the diff. of lat. is the greater.

Second Rule: Length $= \sqrt{(\text{departure})^2 + (\text{diff. of lat.})^2}$.

This is perhaps quicker if only the length is required.

Example: Find the length of AB in the last example.

By rule 1, length $= 943{\cdot}1 \text{ cosec. } 84^\circ\ 2'\ 36'' = 948{\cdot}2$.

By rule 2, „ $= \sqrt{943{\cdot}1^2 + (98{\cdot}4)^2} = 948{\cdot}2$.

(*e*) Given the bearings of AB and BC (fig. 63) and the co-ordinates of A and C, it is required to find the lengths of AB and BC.

First Rule: Find the length and bearing of AC as above, then find the angles CAB, ABC, and BCA from the bearings as in (*b*) above. Then solve the triangle ABC from one side and three angles.

Second Rule: Get the reduced bearings (say α and β respectively), and cardinal points of AB and BC. Then let x = length of AB, y = length of BC.

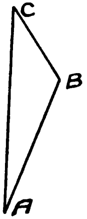


FIG. 63.

$$\text{Then } x \cos \alpha = \text{diff. of lat. A to B}$$
$$\text{and } y \cos \beta = \text{,,} \quad \text{,,} \quad \text{B to C.}$$

Either of these is to be reckoned *minus* if the first cardinal point for that line is *south.*

Then the sum of these must give the total northing from A to C.

Hence $x \cos \alpha + y \cos \beta =$ N. co-ord. of C − N. co-ord. of A.
Similarly $x \sin \alpha + y \sin \beta =$ E. co-ord. of C − E. co-ord. of A.

The latter equation is obtained from the departures, and either of the terms on the left is negative if the second cardinal point for that line is W.

Example: Bearing: AB = 29° 12′; BC = 327° 51′
Co-ordinates: A = N. zero; E. 200·0
C = N. 1716; E. 27·0.

R.B. of AB = 29° 12′ N.E.; R.B. of BC = 32° 9′ N.W.

∴ Both diffs. of lat. are positive, but the departure is negative for BC, as the second cardinal point is W.

∴ if x = length of AB, y = length of BC

$$x \cos 29^\circ 12' + y \cos 32^\circ 9' = 1716 - 0 = 1716$$
$$x \sin 29^\circ 12' - y \sin 32^\circ 9' = 27 - 200 = -173.$$

Solving simultaneously, $x = 970{\cdot}1$; $y = 1026{\cdot}6$

Accurate Traverses.—Such cases arise particularly in ranging railway and other curves round hillsides, and in tunnelling work, especially for sewers or tube railways in towns. In these cases it is often impossible to close the traverses conveniently, and generally very high accuracy is required in order that tunnelling may be started at more than one point, and that the different tunnels may meet properly.

Great care must therefore be taken to centre exactly over the station, to observe to the bottom of the signal (which should be an arrow held carefully in position whilst being observed), or, if this be impossible, the signal should be carefully "plumbed" before being observed. Two or three "repetitions" of each angle should be taken, and both verniers should be read in each case. If there is much difference of level the angles should also be read with both "faces" of the instrument, as will be described later (p. 161), and the final mean taken.

The distances should be measured with a steel tape, the length of which, at a known temperature and under a known tension, should be found by comparison with a standard immediately before beginning the survey. In use, it is desirable that the same tension should be applied, and the temperature should be taken. The tape should be laid flat, both during standardization and in use.

Many methods have been adopted for applying the tension, and for transferring the end of the tape to the ground. Unless the lengths are very great it is sufficient to apply the tension by a spring balance, a loop of strong string, ACB, being attached to the handle as shown. The spring balance is hooked into the string at C, so that the handle can be pressed down flat. The writer generally uses an old chisel, with its back held vertically against the handle between A and B, to transfer the end to the ground, arrows being hardly accurate enough. Once the chisel is in position, the method of marking depends upon whether the marking is to be done on a wooden peg, or to be ruled say on a stone pavement, but it is to be remembered that the *back* of the chisel marks the end of the chain.

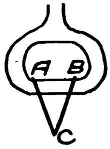


FIG. 64.

If a line be ruled close up against it with a fine pencil, the front of this line is the mark.

Correction for Slope.—If there is any slope, the level should be taken at each mark, and a correction for slope applied.

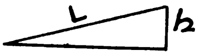


FIG. 65.

Let L be the nominal length of the chain, and h be the rise or fall in any chain length, both in feet, h supposed small in comparison with L.

Then for one chain along the slope, the correct horizontal distance will be $\sqrt{L^2 - h^2} = (L^2 - h^2)^{\frac{1}{2}}$. This can be written $L\left\{1 - \left(\frac{h}{L}\right)^2\right\}^{\frac{1}{2}}$ and, as the fraction $\frac{h}{L}$ is small, if we expand by the binomial theorem, we obtain $L\left\{1 - \frac{1}{2}\left(\frac{h}{L}\right)^2\right\} = L - \frac{h^2}{2L}$.

Thus as the measured length is L, the *correction* on that chain will be $-\frac{h^2}{2L}$.

If L = 100 feet, the value of this fraction for each chain length is easily calculated from the known value of h, by the aid of a table of squares, and tabulated. The results are then summed, and subtracted from the apparent measured length. In computing these corrections, any slight variation in the length of L, from standard or nominal length, may be neglected.

Other Corrections.—To correct for an error in the length of tape. Let L = nominal length, say 100 feet, l = actual length at temperature and tension used in comparison with standard. Then the correction *per chain* is $l - L$. This will be *plus* or *minus* according to whether l is greater or less than L. It is multiplied by the number of measured chains, and then added or subtracted according to sign.

To correct for temperature, let t_1 = temperature at time of standardization, t_2 = temperature in use, both in *Centigrade* degrees. Then correction per chain = $0{\cdot}000012\text{L}\ (t_2 - t_1)$. This will be *plus* or *minus* as before, and is to be multiplied by the number of measured chains, and added or subtracted according to sign. 0·000012 is taken as the coefficient of linear expansion of the steel. If the temperatures are in the Fahrenheit scale, the formula becomes $0{\cdot}000007\text{L}\ (t_2 - t_1)$.

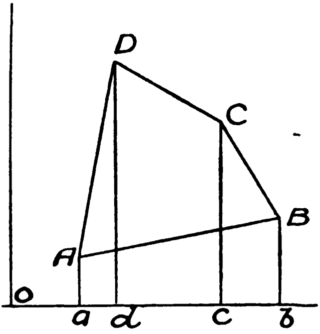


FIG. 66.

Work to be Done at Night.—The work should be done at night as far as possible, both to avoid interference by traffic and to ensure more steady atmospheric conditions.

Areas from Co-ordinates.—If a traverse be closed, the areas within the lines may be found from the co-ordinates as follows: referring to fig. 66, it is evident that the area of ABCD = A*ad*D + D*dc*C + C*cb*B − B*ba*A.

$$\text{Now area of AadD} = \tfrac{1}{2}ad \times (\text{A}a + \text{D}d)$$
$$= \tfrac{1}{2}(\text{O}d - \text{O}a)(\text{A}a + \text{D}d)$$

and similar formulæ may be made up for the other figures.

Therefore area ABCD

$$= \tfrac{1}{2}\{(\text{O}d - \text{O}a)(\text{A}a + \text{D}d) + (\text{O}c - \text{O}d)(\text{C}c + \text{D}d) + (\text{O}b - \text{O}c)(\text{B}b + \text{C}c) - (\text{O}b - \text{O}a)(\text{A}a + \text{B}b)\}$$
$$= \tfrac{1}{2}\{\text{O}d(\text{A}a - \text{C}c) + \text{O}a(\text{B}b - \text{D}d) + \text{O}b(\text{C}c - \text{A}a) + \text{O}c(\text{D}d - \text{B}b)\}$$

by multiplying out and cancelling like terms.

This rule can be put into words as follows :—

Take each east co-ordinate in turn, and multiply it by the difference between the north co-ordinates of the two adjacent points, one on each side. Then sum the results and divide by two.

The differences between the north co-ordinates must be taken always in the same order. Thus suppose we begin with the east co-ordinate of B, namely O*b*.

This is to be multiplied by the difference between the north co-ordinates of A and C. But this difference may be taken as A*a* – C*c*, or C*c* – A*a*. It is usual to take the first difference in such an order as to make the result positive, but once this is settled all other differences must be taken in the same order, whether the results are plus or minus, and the sign must, of course, be attended to.

Thus if we take C*c* – A*a*, then for O*c* we must take D*d* – B*b*, and for O*d* we must take A*a* – C*c*, and so on. If the final result is negative, simply alter the sign to plus.

Example.—

Point.	A.	B.	C.	D.
N.	200	312	681	876
E.	200	962	719	287

Find the area within the lines.

Point.	E. Co-ordinate.	Adjacent N. Co-ordinates.	Difference.	Product.	
				+	−
A	200	876 312	+ 564	112,800	
B	962	200 681	− 481		472,722
C	719	312 876	− 564		405,516
D	287	681 200	+ 481	138,047	
				250,847	878,238
				+	250,847
				2)	627,391
				Ans.	313,695

The answer will be in square feet or square links, according to the units.

It will be evident that with a four-sided figure the actual calculation can be shortened, but it has been worked out as if for a greater number of sides.

The area between the lines and the boundaries can be found from the field book as described on p. 55.

EXERCISES FOR REVISION.

1. Why do we plot a traverse by co-ordinates rather than by scale and protractor? Given the bearing of the first line and all the included angles and lengths, give the rules for calculating the co-ordinates.

2. The bearing of a line AB is 172° 30′ E. of N. The clockwise angle ABC is 328° 47′, and the length of BC is 178·5 feet. The co-ordinates of B, in feet, are 2020 N., and 198·6 E. Find (*a*) the bearing of BC; (*b*) the difference of latitude and departure B to C; (*c*) the co-ordinates of C.

3. Calculate the co-ordinates for the six-sided figure shown in fig. 67.

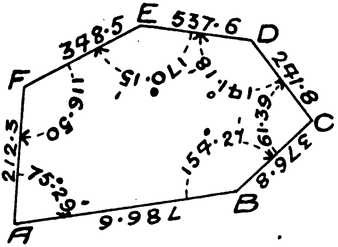


Fig. 67.

Take the bearing of AB as 80° 0′, and the co-ordinates of A as zero.

4. Calculate the area in acres, etc., within the traverse lines from the co-ordinates, all lengths being in links (Gunter's).

5. In the table of co-ordinates on p. 89 find the lengths and bearings of AB and BD, and the clockwise angle ABD.

6. Solve the following example, referring to fig. 68.

A, B, C, D are four points in a town (fixed on the ground and all invisible from one another) through which the centre line of a storm water sewer (to be laid by tunnelling) is to pass in plan.

The sewer is to run straight from A to B and from D to C, and these straight portions are to be connected by a curve EF as shown.

It is desired to sink a shaft at one end of the curve, and it is found, by means of a rough plan, that a radius of 600 feet will bring the end E on to some vacant ground, and that E will be invisible from C, but visible from a point G, 100 feet from C along CL.

A traverse is run along the roads, and the angles and distances shown are obtained.

Taking A as the origin of co-ordinates, and the bearing of AH as zero (and working to one decimal of a foot and to the nearest minute only), you are required to calculate :—

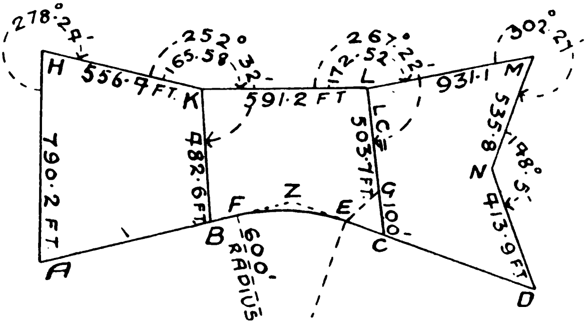


FIG. 68.

(*a*) The co-ordinates of all points except G, E, F, and Z.

(*b*) The bearings of AB and DC.

(*c*) The angles HAB, NDC, and CZB (required for setting out the directions of AB and DC), and the distances AB, DC.

(*d*) The distances BZ and CZ.

(*e*) The tangent distance ZF or ZE, and the distances BF and CE.

(*f*) The co-ordinates of G and E, and the bearing of GE.

(*g*) The angle CGE and the distance GE (required for fixing the position of E). (London B.Sc. Eng. Exam.)

CHAPTER III.

LEVELLING AND CONTOURING.

General Procedure.—In determining the relative levels of points on the earth in surveying, it is usual to imagine a horizontal or *level* surface, usually chosen so as to be at a lower level than the points being dealt with. This is called the *datum*, or for small areas, the *datum plane*. The levels of the various points are then expressed by stating their heights above this datum.

These heights above the datum are called the *reduced levels* of the points.

In Great Britain the datum chosen for the levels of the Ordnance Survey is a level surface passing through a mark on the base of a lighthouse at the mouth of the Mersey, giving the mean level of the sea during a certain period.

In Ireland there is a similar mark on a lighthouse at the mouth of the Liffey.

Thus all reduced levels marked on the ordnance maps are above mean sea level, so far as the latter can be easily determined. In this country levels are usually stated in feet.

Reduced levels are shown in various ways, for example :—

(*a*) By *spot levels* (fig. 69). The reduced levels of the points are marked on the plan in figures. Thus in fig. 69 A is higher than B by 30·9 feet, and the datum is lower than B by 71·8 feet.

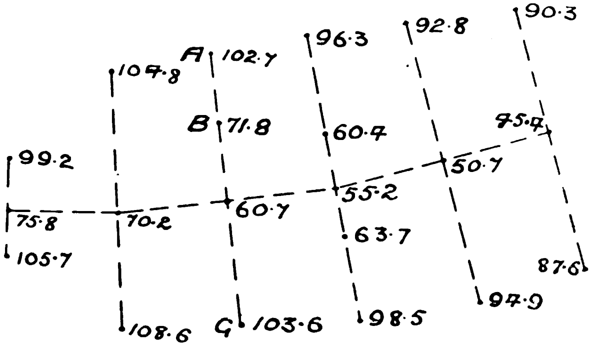


FIG. 69.

Examples of spot levels will be found along roads, etc., on, for instance, the 25-inch ordnance maps of England.

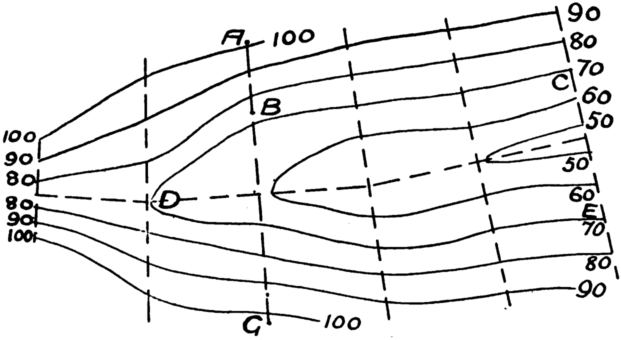


FIG. 70.

(*b*) By *contours*. In fig. 70 the same portion of a valley is shown as in fig. 69. Contours are lines drawn through all points on the surface *whose levels are the same*, and they are generally drawn to show all points at some round number of feet above the datum. In fig. 70 they are drawn for every ten feet from 50 to 100, the levels being written at the end of each contour. The students should compare fig. 69 and fig. 70 to see that they agree. Thus all points on CDE are at a level of 70 feet above datum. Examples of contours are shown, for instance, on the 6-inch ordnance maps of England.

(*c*) By *sections*. In drawing a section the surface of the earth is supposed to be cut vertically along some definite line (straight or curved), and the reduced levels are shown for the points along that line only. Fig. 71 is a section along the line ABG of fig. 69 or 70. The horizontal distances are the same as on the plan (thus *ag* is the same length as AG in fig. 69). The line *abg* represents the datum plane, and the heights *a*A, etc., represent the reduced levels on some definite scale.

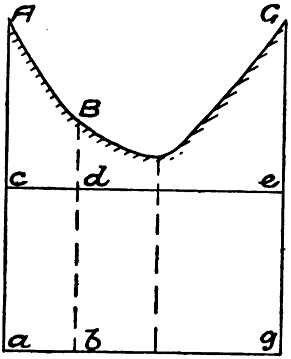


Fig. 71.

Usually in drawing sections the "vertical scale" (that is the scale of reduced levels) is greater than the horizontal. Thus with a horizontal scale of 1 chain to 1 inch, or 100 feet to 1 inch, we often use a vertical scale of 10 feet to 1 inch.

Also, if all the reduced levels are high, we generally plot a section *not* from datum, but from a plane parallel to datum and some round number of feet above it. Thus in fig. 71, we might have started with the line *cde* 50 feet above datum. Then subtract 50 from each reduced level. This enables a bigger vertical scale to be used.

If the line along which the section is drawn is curved in plan the section is drawn as if the line were stretched out straight, so that its length is equal to the whole length of the curved line.

The Level.—The instrument most commonly used for determining levels is the Dumpy level (fig. 72).

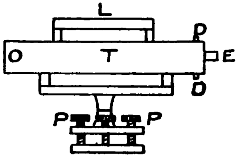


FIG. 72.

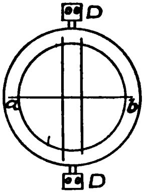


FIG. 73.

It consists, as will be evident, of a telescope T above which is a spirit level L. This enables the telescope to be brought truly horizontal by means of the plate screws P underneath.

Just inside the eyepiece E of the telescope is the diaphragm (fig. 73), supported by the diaphragm screws D, by which it can be raised or lowered.

It consists of a ring of metal across which are stretched three fine threads—usually spider's webs—as shown in fig. 73. The thread *ab* is truly horizontal, and if all points on it be joined to the optical centre of the object glass (O, fig. 72) and the rays so drawn be produced outwards, they will all lie in one plane, which is called the *plane of collimation.*

The Staff.—The level is used in combination with a levelling staff (fig. 74), graduated from the bottom upwards. The form shown is known as the Hopwith staff, and is the most generally used. But many other systems of graduation are in vogue, particulars of which can be obtained from the makers. In the opinion of the writer one system of graduation is just about as good as another. The common lengths of staff are from 10 to 14 feet.

Taking Levels.—Let ABCDE (fig. 75) represent a section of the ground to be levelled. The level is set up at X, preferably on one side of the line, and *levelled* by the plate screws in two directions. The method of doing this is the same as for the theodolite (p. 65).

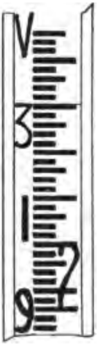


FIG. 74.

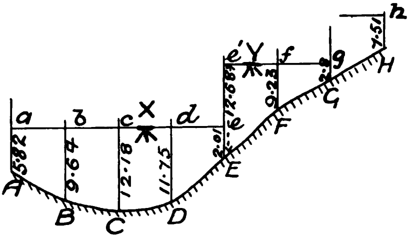


FIG. 75.

The plane of collimation then becomes horizontal, and is represented by *abcde* in fig. 75.

The staff is then held at each of the points A, B,. etc., in turn, the diaphragm webs are focussed as described for the theodolite (p. 65), and on looking at the staff and focussing it with the focussing screw, we see an inverted image of the staff, and we can read the division at which the staff appears to be intersected by the horizontal hair.

The readings supposed to be obtained are entered on the diagram. As the staff is graduated from the bottom upwards, these readings tell how far the various points are *below* the horizontal plane *abcde* of collimation, or conversely, how far the latter is above each point.

Thus it is evident that if the reduced level of any one point (say A) be known, the rest can be calculated from the readings.

On arriving at E, the staff cannot be carried to F without going above the collimation plane, so that it would be invisible. Hence, to carry on the levels, we keep the staff at E, carry the level on to a new position Y, and take a second reading (a *backsight*) on the staff at E. This enables us to carry on from E to G, and so on.

The *first reading* from any one position of the level is called a *backsight*. The *last* is called a *foresight* and the rest are *intermediates*.

The horizontal distances between the points are measured while the work proceeds, either by *stepping* or by making the hypotenusal allowance (p. 34) if the slope is sufficient.

Bench Marks.—Levels in engineering practice are generally taken for the purpose of some proposed work. In carrying out this work, all points lying on the line of the original section will probably be either excavated or covered over. Hence it is obviously desirable to have some fixed points, at intervals, *near* but not *on* the site of the proposed work. The reduced levels of these points being known, they can be used as points of reference for levels during the progress of the work. Such points are called *bench marks*. They may be chosen in any convenient positions such as corners of door-steps, etc., the essential points being that they must not be liable to change in level, and that their position must be capable of being described so that they can be identified with certainty. It is usual for the levelling to begin at a bench mark, and frequently the datum is taken some round number of units below this. The chainage of distances, of course, begins at the commencement of the section.

In the ordnance survey of England the bench marks were specially cut—usually in the sides of buildings, etc.—in the shape of a broad arrow (fig. 76), and their levels above ordnance datum are stated on the maps.

The figure given on the map refers to the level of the horizontal line.

In engineering work bench marks should not be more than about half a mile apart, or less if the slope is steep. Ordnance bench marks should be chosen when possible, as their known levels give an independent check on the accuracy of the work.

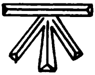


Fig. 76.

Checking the Work.—When the section is completed, the work is checked by levelling back to the starting-point, doing no chaining, not necessarily following the same route, and taking backsights and foresights only. The reduced level at the end should then come back to the original figure, or nearly so.

These levels are marked, in the remarks column of the book, as "check levels," to show why they were taken. No use is made of them in plotting.

Turning Points.—A point such as E (fig. 75) at which the staff is held while the level is carried forward is called a "turning point," "change point," or "change sight point". Turning points may be at definite points on the section, as in this case, but in town surveying, for instance, where the levels are required say along the centre line of a road for a sewerage scheme, such points would be inconvenient positions in which to hold a staff whilst the level is moved and adjusted at a new station. Hence it is better, when all the required readings have been taken from any position of the instrument, to send the staff-holder to some quiet spot off the line, giving him a pre-arranged signal to show that you require a foresight reading, when he must be trained to look for a suitable spot where he will not be interfered with by traffic (if also suitable for a bench mark so much the better) and the staff is held there. The foresight and backsight are entered as usual, the distance column is left blank, and the letters T.P. (for turning point) entered in the remarks column, to show why the readings were taken. If used for a bench mark, add "B.M. on N.E. corner of door-step," or other description.

Methods of Booking.—Returning now to fig. 77, which is a reproduction of fig. 75, we will assume that the first reading (not shown on the fig.) was 0·84 on a B.M., that the horizontal distances from point to point are each one chain, and that, after the reading at G, a foresight and backsight (of ·76 and 12·84 respectively) were taken on a turning point not shown in the figure.

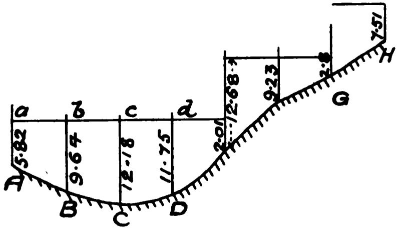


Fig. 77.

We will choose our datum plane 20 feet below the first bench mark, so that the reduced level of the latter is 20·00.

Then the readings are booked and worked out as follows:—

Back-sight.	Inter-mediate.	Fore-sight.	Collimation Level.	Reduced Level.	Dis-tance.	Remarks.
·84			20·84	20·00	—	B.M. on post near A.
	5·82			15·02	0·00	Pt. A on plan.
	9·64			11·20	1·00	
	12·18			8·66	2·00	
	11·75			9·09	3·00	
12·68		2·01	31·51	18·83	4·00	
	9·23			22·28	5·00	
	2·80			28·71	6·00	
12·84		·76	43·59	30·75		T.P.
	7·51			36·08	7·00	Pt. H.
·94		13·63	30·90	29·96		Check levels.
		10·88		20·02		Pt. A.
27·30		27·28				

In this system of booking, the heading "collimation level" is sometimes replaced by the words, "height of instrument," or the letters H.P.C., meaning "height of the plane of collimation". In any case, the column gives the reduced level of the plane of collimation, *abcd*. It is evident that if a reading be taken on a point whose reduced level is known, we must *add* the reading to the known reduced level, to get the collimation level. Conversely, knowing the collimation level we must *subtract* the reading to get the reduced level. Hence we add the backsight 0·84, to the level of B.M. (20·00) to get the first collimation level (20·84), that is, the level of the plane *abcd*. Then the successive readings from that position of instrument are subtracted from this to get the various reduced levels, finishing with the foresight (2·01) which gives the reduced level (18·83) of the first turning-point. Then *add* the new backsight (12·68) to this to get the new collimation level, and so on, *adding backsights and subtracting all other readings*.

We work back to 20·02 at the end, showing an error of 0·02 foot, which would generally be of no importance.

To check the arithmetic, we add up the backsights and add up the foresights. The difference between the results should agree with the difference between the *first and last reduced levels*. In this case 27·30 − 27·28 = $\underline{0{\cdot}02}$, as before.

In the table below, the same readings are entered, but worked out by the "rise and fall" method.

Back-sight.	Intermediate.	Fore-sight.	Rise.	Fall.	Reduced Level.	Distance.	Remarks.
·84					20·00	—	B.M.
	5·82			4·98	15·02	0·0	Pt. A.
	9·64			3·82	11·20	1·0	
	12·18			2·54	8·66	2·0	
	11·75		0·43		9·09	3·0	
12·68		2·01	9·74		18·83	4·0	
	9·23		3·45		22·28	5·0	
	2·80		6·43		28·71	6·0	
12·84		·76	2·04		30·75	—	T.P.
		7·51	5·33		36·08	7·0	Pt. H.
26·36		10·28			20·00		
10·28					16·08		
16·08							

In this case, the check levels have been omitted, and the reading at H is regarded as the last. It will be obvious from the figure that, if any two readings be taken with one position of the level, the difference between them will tell the rise or fall between the corresponding points. If the *second* reading is the greater, there has been a *fall*, and *vice versa*. Thus 5·82 – 0·84 = 4·98, gives the fall from the B.M. to pt. A ; 9·64 – 5·82, gives the fall from A to 1·00, and so on down to 11·75 – 2·01 = 9·74, which is the rise from 3·0 to 4·0 (a *rise* because the second reading is *less* than the first).

Then the level is moved, and the rise from 4·0 to 5·0 is found by subtracting 9·23 from 12·68, and so on. The reduced levels are then worked out by adding rises and subtracting falls from point to point, as shown in the table.

The arithmetical check is shown in the table by the same rule as before.

Comparison of Methods.—When there are many intermediates, the rise and fall method involves more arithmetic, as two distinct operations are required for each reduced level. But the advantage of it is that, as each reduced level is obtained from the one above, any arithmetical error anywhere will be carried forward, and will be detected in the check at the end. Whereas, by the collimation level method, each intermediate point is worked out independently, so that an arithmetical error in one of these might escape detection.

The writer personally thinks that the rise and fall method is preferable for those who are pretty accurate arithmeticians, and who will seldom, in consequence, find it necessary to do their calculations twice over, because it supplies a security against occasional errors.

But if the calculations have, as a rule, to be repeated, this repetition is in itself a check, and it is doubtful if the extra labour involved in the rise and fall method is then justifiable.

From Page to Page.—The arithmetical check should be shown *on each page* of the book, as in the examples. If the last reading on a page is an intermediate, enter it nevertheless in the *foresight* column.

Then carry this same reading forward to the *backsight* column to begin the new page. Carry forward also its reduced level, and write "brought forward" in the remarks column. Then proceed as before.

If the last point on the page is a turning-point, enter the *foresight only* on that page. Work out the reduced level and carry it alone forward, but write the new backsight on the new page, in the backsight column, and opposite the reduced level which has been carried forward. Thus each page begins with a backsight and ends with a foresight.

Choosing Positions of Level.—*The* point in choosing the position for the instrument is that one should be able to read the backsight, and then as far as possible along the line of the section. Experience will best enable one to accomplish this, but the beginner should *make sure* that he can read at least the backsight before finally levelling up in the position chosen. To do this, lift *one* leg of the tripod off the ground, and, by moving it about, both the main level and the small cross level can be brought fairly central, then press that leg down, by a turning action, without moving the instrument at all scarcely. Then *take* aim at the staff, *over the top* of the telescope, bring the bubble central by that pair of opposite plate screws which *most nearly* agree with the direction of the telescope, and *then* look through to see if the staff is visible. If we are too high or too low we must move accordingly. If the reading is about right—say within a foot or eighteen inches of the top or bottom of the staff, as the case may be—we press the legs down firmly, and level up permanently, and then read the backsight again.

The level should be set up so that the sun, if shining, will be on the observer's *back* as far as possible, as it is often difficult to read with the sun shining into the telescope.

Parallax.—If the focussing is properly done, the image of the staff will be in absolute coincidence with the hair *ab*, and no movement of the observer's eye will cause the reading to vary.

But if not the hair may be at *a*, as shown in section in fig. 78, while KL is the image of the staff, which, of course, is formed by the object glass, inside the telescope.

If now the eye of the observer (which is looking at this image through the eyepiece) be moved up and down, say from M to N, it is evident that the hair will appear to move up and down the image, through a short distance, as shown by the dotted lines.

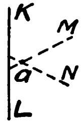


FIG. 78.

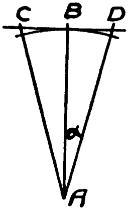


FIG. 79.

This faulty adjustment is called *Parallax;* it must be tested for by moving the eye, and, if the hair appears to move, touching the focussing screw till all movement ceases. This has the effect of altering the relative positions of the hair and image, so as to bring them into coincidence.

Of course this assumes that the eyepiece has previously been adjusted to give the best view of the hairs (see p. 65).

Verticality of Staff.—It is obviously necessary that the staff should be held vertically, especially if the reading be at all near the top.

Thus if a staff be held at A (fig. 79) its proper direction is AB. But if it be held in the position of AC or AD, and if CBD be the plane of collimation, then the reading obtained will be represented by AC or AD as the case may be. Now let a = angle from vertical. Then AD = AB sec a. Hence the error in the reading = AD − AB = AB (sec a − 1), and therefore increases proportionally to AB.

To enable the staff-holder to see if the staff is vertical, many devices have been used, of which the most common are the plumb-bob AB (fig. 80), suspended at the back of the staff, so that when the latter is vertical the bob hangs directly over an index mark on a bracket D, or one or more spirit levels, C, attached to the side of the staff, so that they are central when the staff is vertical. If the staff is not fitted with these, or if the staff-holder cannot be depended upon, the surveyor should send a special signal, indicating that the staff-holder is to wave the staff slowly backwards and forwards. On receiving this signal, the staff-holder must *take one short step backwards*, slope the staff back to his body, then slowly forwards, almost to arm's length, to ensure that it goes to each side of the vertical.

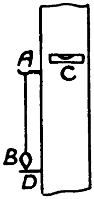
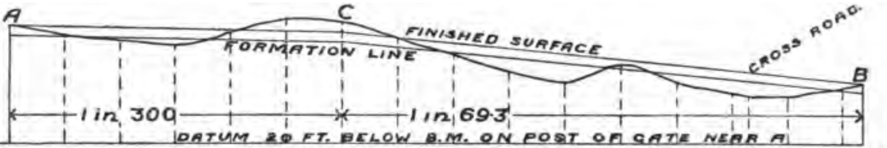


FIG. 80.

The surveyor then takes the *lowest reading*, which will correspond with the vertical staff AB (fig. 79).

Any *side* slope of the staff can be detected by the two vertical hairs in the diaphragm, and the observer gives a special signal to show which way it is to be moved.

The staff-holder should be trained to stand with one foot on each side of the staff, toes touching the



DISTANCES (IN CHAINS)	0	1	2	3	4	5	6	7	8	9	10	11	12	13	13·3	14	15	15·36
REDUCED LEVELS (FT)	21·07	19·40	18·46	17·40	19·83	22·17	21·68	18·82	15·75	12·82	10·94	14·24	10·02	9·21	8·82	8·62	10·15	10·83
FORMATION LEVELS.	19·32	19·10	18·88	18·66	18·44	18·22	18·00	17·05	16·09	15·14	14·19	13·23	12·28	11·33	11·04	10·38	9·42	9·08
DEPTHS OF CUTTING	1·75	·30			1·39	3·95	3·68	1·77				1·01					·73	1·75

sides of the staff, the back of the staff resting against his chest or abdomen, and one hand *on each side* of the staff, hands open and palms firmly pressed against the side of the staff, elbows down and pressed against the body for steadiness. It is obvious that, from this position, he must step back before waving the staff.

Care at Turning-points.—For ordinary work it is sufficient to read intermediate sights to the nearest tenth of a foot. But backsights and foresights should be read to the nearest hundredth, as any error in these readings is carried forward and affects the whole of the rest of the survey.

Hence special care must be taken at turning-points to see that the bubble is central (if not, bring it central with the nearest plate screws before reading) to ensure that the staff is vertical, and so on. Hence it is always well to send the staff-holder a prearranged signal to let him know when a reading is to be a foresight. This not only puts him on the *qui vive*, but intimates that he must make sure that his staff is on solid ground. Metal base plates can be obtained (the suitable shape can be seen in the catalogues of most makers) for him to place on the ground and ram down to rest the staff on in soft ground.

Finished Sections.—When the section is completed, assuming that it is intended for a road or railway, the proposed work may involve excavation and embankment.

The proposed gradients AC, CB (fig. 81) are chosen in accordance with engineering principles which would be out of place here, and are usually shown by two *red* lines, the upper one denoting the finished surface line, while the lower is the "formation" line, being the line to which the ground is to be levelled off to receive the ballast, broken stone, or whatever it may be, of which the

LEVEL BOOK FOR FIG. 81.

Back-sight.	Inter-mediate.	Fore-sight.	Collima-tion Level.	Reduced Level.	Dis-tance.	Remarks.
4·18			24·18	20·00	—	B.M. on post near A.
	3·11			21·07	0·0	Point A.
	4·78			19·40	1·0	
	5·72			18·46	2·0	
	6·70			17·48	3·0	
	4·35			19·83	4·0	
	2·01			22·17	5·0	
0·37		2·15	22·40	22·03	—	T.P.
	0·74			21·66	6·0	
	3·58			18·82	7·0	
	6·65			15·75	8·0	
	9·58			12·82	9·0	
	11·46			10·94	10·0	
	8·16			14·24	11·0	
	11·48			10·92	12·0	
	13·19			9·21	13·0	
5·64		13·78	14·26	8·62	13·3	Centre of cross road.
	5·74			8·52	1·0	On cross road to left.
	5·79			8·47	2·0	On cross road to left.
	4·36			9·90	1·0	On cross road to right.
	5·64			8·62	14·0	On proposed line.
	4·11			10·15	15·0	
	3·43			10·83	15·36	Point B.
8·63		1·64	21·25	12·62	—	Check levels.
5·72		2·63	24·34	18·62	—	
		4·31		20·03	—	On B.M.
24·54		24·51		·03		
24·51						
·03						

permanent way is to be formed. Thus the formation line represents the line to which the top of the embankments or bottom of cuttings must be brought, and its distance below the finished surface line is equal to the total thickness of material of which the way is to formed.

In road and railway work absolutely uniform gradients are relatively unimportant, and the formation levels and depth of earthwork are often simply measured from the drawing. But in sewer work, for instance, it is important that the gradients should be exactly uniform from point to point, and, to ensure this, the level of the "invert" of the sewer (or the bottom of the trench) should be *calculated* at all intermediate points. The depth of trench at each point is then found by subtracting the figure so found from the reduced level of the ground at the same point.

As an example of this calculation, we will work out the formation levels and depths of earthwork for the section shown in fig. 81.

It is assumed that the finished surface is level with the ground at A and B, that the depth from finished surface to formation is 1·75 feet, and that at the point C where the chainage is 6 chains, the formation level has been chosen as 18·00. Then working from C to B we have :—

$$\begin{aligned} &\text{Formation level at C} &&= 18{\cdot}00 \\ &\text{Formation level at B} = 10{\cdot}83 - 1{\cdot}75 &&= \underline{9{\cdot}08} \\ &&\text{Fall} &= \underline{\underline{8{\cdot}92}} \text{ feet.} \end{aligned}$$

$$\begin{aligned} \text{Distance} &= \text{chainage of B} - \text{chainage of C} \\ &= 15{\cdot}36 - 6{\cdot}00 = 9{\cdot}36 \text{ chains.} \end{aligned}$$

$$\therefore \text{ fall per chain} = \frac{8{\cdot}92}{9{\cdot}36} = \underline{{\cdot}953 \text{ foot.}}$$

Then starting from 18·00 at C, we must subtract 0·953 foot for each chain to find the successive formation levels, thus :—

$$\begin{array}{lrcr}
\text{F.L. at} & 6{\cdot}0 & = & 18{\cdot}00 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 7{\cdot}0 & = & 17{\cdot}047 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 8{\cdot}0 & = & 16{\cdot}094 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 9{\cdot}0 & = & 15{\cdot}141 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 10{\cdot}0 & = & 14{\cdot}188 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 11{\cdot}0 & = & 13{\cdot}235 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 12{\cdot}0 & = & 12{\cdot}282 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 13{\cdot}0 & = & 11{\cdot}329 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 14{\cdot}0 & = & 10{\cdot}376 \\
 & & & {\cdot}953 \\ \cline{4-4}
,, & 15{\cdot}0 & = & 9{\cdot}423 \\
 & & & {\cdot}343 \\ \cline{4-4}
,, & 15{\cdot}36 & = & \underline{\underline{9{\cdot}080}} \ \text{which checks.}
\end{array}$$

The 0·343, which is subtracted at the end is the fall for 0·36 chain. It is found by taking ·36 of the fall per chain; thus 0·953 × 0·36 = 0·343. The final level, 9·08, checks with the formation level previously found at B.

The depth of earthwork is found at each point by taking the difference between the formation and reduced levels. It is *cutting* if the latter is the greater, and vice versa.

The figures from A to C are worked out in the same way, and are left as an exercise for the student. To find the formation level at the cross roads, the chainage is 13·30. Hence we take fall per chain × 0·30 = 0·953 × 0·3 = 0·286, and subtract this from the formation level at 13·00, which is taken from the table; thus 11·329 − 0·286 = 11·043.

This, like all the others, is written down as 11·04, to the nearest hundredth only, but we calculate to three figures to avoid accumulated error.

Calculation of Earthwork.—In calculating the volumes of earthwork, many different methods are used.

It is necessary to know exactly what the cross-section will be in cutting and in embankment.

For instance in fig. 82 the side slopes are carried down to the formation level AB, and on each side there is a ditch and a small earth bank, as shown. It is, perhaps, most usual to calculate volumes for the section whose shape is ABCD, in the first place, and to allow for the ditches and banks afterwards if desired.

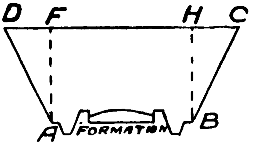


Fig. 82.

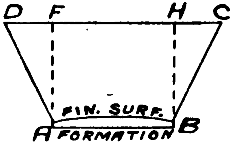


Fig. 83.

In fig. 83, which is a possible example for a short cutting, the slopes are carried down to about the finished surface level, and the part below is excavated vertically and filled in again as shown.

In this case, the depths used in calculating the volumes should be taken down to the finished surface only, the volumes calculated for sections like ABCD, and the volume of the part below worked out separately and added.

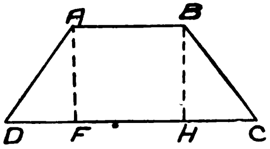


Fig. 84.

Fig. 84 shows a cross-section of an embankment, without any earth banks at the sides, or other additions.

The width AB is, in all cases, called the *formation width.*

The side slopes are fixed according to the material, in accordance with engineering principles. They are expressed as, say, 2 to 1, meaning 2 *horizontal* to 1 *vertical.*

Thus with a slope of 2 to 1, CH in fig. 84 would be BH × 2, or CH : BH : : 2 : 1.

We will assume that the formation width is 30 feet, both in cutting and embankment, and the side slopes 2 to 1 in embankments and $1\frac{1}{2}$ to 1 in cuttings, and that the side slopes are carried to formation level.

Then to work without tables, we must find an expression for the area ABCD.

Let the depth BH (figs. 82 to 84) $= h$

$AB = b$, and side slopes n to 1.

$$\begin{aligned} \text{Then ABCD} &= \text{ABHF} + 2 \times \text{BHC}. \\ &= b \times h + 2 \times \tfrac{1}{2} \times \text{BH} \times \text{HC}. \\ &= bh + h \times nh, \text{ since HC} = n \times \text{HB}. \\ &= bh + nh^2. \end{aligned}$$

Thus in fig. 85 if Bb, Dd and Ff be the depths of a cutting at known points, and if C and E be the middle points of BD and DF respectively, then to find the volume between C and E, we must find the area corresponding to the depth Dd, and multiply by the length CE. Alternatively, the *mean* of the areas Bb and Dd, multiplied by BD, will give the volume from B to D.

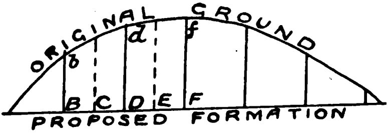


FIG. 85.

Both formulæ are approximate merely. The second is probably rather more accurate.

Thus suppose $h_1, h_2 \ldots h_m$, are the depths of cutting at known points, $l_1, l_2 \ldots l_{m-1}$ the distances between these known points; b the formation width, and side slopes n to 1.

Then volume

$$= l_1 \left\{ \frac{b\,(h_1 + h_2)}{2} + \frac{n\,(h_1^2 + h_2^2)}{2} \right\} + l_2 \left\{ \frac{b\,(h_2 + h_3)}{2} + \frac{n\,(h_2^2 + h_3^2)}{2} \right\}$$

$$\ldots + l_{m-1} \left\{ \frac{b\,(h_{m-1} + h_m)}{2} + n \, . \, \frac{(h_{m-1})^2 + (h_m)^2}{2} \right\}.$$

If all the distances are equal, so that $l_1 = l_2$, etc., this becomes volume

$$= l_1 \left\{ b \left(\frac{h_1}{2} + h_2 + h_3 \ldots + \frac{h_m}{2} \right) + n \left(\frac{h_1^2}{2} + h_2^2 + h_3^2 + \ldots + \frac{h_m^2}{2} \right) \right\},$$

and the calculations are tabulated as follows, referring to the cutting from four chains to seven chains in fig. 81.

Chainage.	Depth.	Do. × Factor.	Square of Depth.	Do. × Factor.
4·00	1·39	·70	1·93	·97
5·00	3·95	3·95	15·60	15·60
6·00	3·66	3·66	13·40	13·40
7·00	1·77	·88	3·13	1·56
		9·19		31·53
		30		1·5
		275·70		31·53
		47·30		15·76
		323·00		47·29

The first two columns are taken from the section. In the third column the first and last depths are *halved*, and the rest remain as in column No. 2, because in the formula we have $\frac{h_1}{2} + h_2 +$ etc.

Column No. 4 is filled in from a table of squares, given in all pocket-books and mathematical tables, and in column No. 5 the first and last of these are *halved* as before.

Sum column No. 3, and multiply by the formation width, b, in this case 30 feet.

Sum column No. 5, and multiply by the ratio, n, of the side slopes, in this case 1·5.

Then add the results as shown; this gives 323·00. This figure is now multiplied by the distance, l_1, between sections, in this case 66 feet or 1 chain.

$\therefore$ volume $= 323{\cdot}0 \times 66 = 21{,}318$ cubic feet, or 790 cubic yards nearly.

Simpson's rule may be used instead, in which case, however, we must have an *odd* number of depths; the factors used for column 3 and column 5 are 1 for the first and last depths, and 4 and 2 alternately for the rest, beginning with 4 for the second one; and the multiplier at the end is $\frac{l_1}{3}$, or 22 feet in this case, instead of l_1. Otherwise the tabulation is the same as with the above formula.

The table shows the volume from 4 to 7 chains. From the section, by measurement, we see that the formation line is in cutting from 3·6 chains to 7·8 chains.

To allow for the volume from 3·6 to 4·0 chains, we take the distance (= 0·4 chain) multiplied by *half* the sectional area at 4 chains. Thus if d be the outside fractional distance, h the height at the known point, volume

$$= \frac{d \times (bh + nh^2)}{2}.$$

The volume between 7·0 chains and 7·8 chains is allowed for in the same way.

$\therefore$ allowance for ends of cutting

$$= \frac{0{\cdot}4 \times 66}{2}(30 \times 1{\cdot}39 + 1\tfrac{1}{2} \times 1{\cdot}93) + \frac{{\cdot}8 \times 66}{2}(30 \times 1{\cdot}77 + 1{\cdot}5 \times 3{\cdot}13)$$

= 2115 cubic feet or 78 cubic yards nearly.

It will be seen that this is equivalent to multiplying the first and last lines in the above table by additional factors to allow for these extra lengths, this additional factor in each case being $\frac{d}{2l}$, where d is the outside length to the end of the cutting, and l the ordinary distance between points of known level.

Thus we have the rule: if the cutting or embankment ends at the points where the levels are known, tabulate as above, using 0·5 for the first and last factors: but, if not, let l be the uniform distance from point to point, d_1 the distance from the commencement of the cutting to the first known point, and d_n the distance from the last known point to the end of the cutting; then the first factor is $0{\cdot}5\left(1+\frac{d_1}{l}\right)$, and the last is $0{\cdot}5\left(1+\frac{d_n}{l}\right)$.

In our case, $l = 1$ chain; $d_1 = 4{\cdot}0 - 3{\cdot}6 = 0{\cdot}4$ chain and $d_n = 7{\cdot}8 - 7{\cdot}0 = 0{\cdot}8$ chain $\therefore \frac{d_1}{l} = {\cdot}4$; $\frac{d_n}{l} = {\cdot}8$, and the complete tabulation is as follows :—

Chainage.	Factor.	Depth.	Do. × Factor.	Square of Depth.	Do. × Factor.
4·00	·7	1·39	·97	1·93	1·35
5·00	1·0	3·95	3·95	15·60	15·60
6·00	1·0	3·66	3·66	13·40	13·40
7·00	·9	1·77	1·59	3·13	2·82
			10·17		33·17
			30		1·5
			305·10		33·17
			49·75		16·58
			354·85		49·75

Here the first factor $= (1 + {\cdot}4) \times {\cdot}5 = {\cdot}7$, and so on.

The volume $= 354{\cdot}85 \times 66 = 23{,}420$ cubic feet.

By the previous tabulation, volume $= 21{,}318 + 2115 = 23{,}433$ cubic feet.

The discrepancy of 13 cubic feet is due merely to contracted arithmetical working.

Augmented Depths Method.—A similar method of working by means of the "augmented depths" is given in Rankine's works, and in "Elementary Practical Mathematics" by the present writer, but the writer considers it less convenient than that given above.

Earthwork Tables.—Assuming that the ground surface is quite smooth, and level transversely, it is clear from fig. 86 that the earthwork between any two cross-sections can be divided into: (1) a wedge-shaped central portion ABCDEFGH whose breadth is everywhere the same, that is, equal to the formation width, AD or EH; the heights at the two ends are equal to the depths of earthwork at those points, viz. AB and EF; (2), the portion excavated for the side slopes, namely the figures CLDGMH and AKBENF. In these, the breadth CL or GM is proportional to the depth, and becomes zero when the depth is zero.

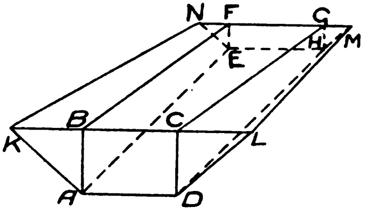


FIG. 86.

Hence these portions are really truncated pyramids, and the formula for the volume of one of them is

$$\frac{l}{3}(a_1 + \sqrt{a_1 a_2} + a_2),$$

where l is the length CG, and a_1 and a_2 are the areas of the triangles CDL and GHM.

Now if $\text{CD} = h_1$, and the slopes are n to 1, then $\text{CL} = n \times \text{CD} = n \, . \, h_1$; and area of $\text{CDL} = h_1 \times \frac{\text{CL}}{2} = \frac{n \, . \, h_1^2}{2}$.

Similarly area of $\text{GHM} = \frac{n \, . \, h_2^2}{2}$, if $\text{GH} = h_2$.

$$\therefore \text{volume} = \frac{l}{3} \times \frac{n}{2}(h_1^2 + h_1 h_2 + h_2^2), \text{ for one side only.}$$

The volume of the wedge-shaped piece is $\frac{l}{2} \times b(h_1 + h_2)$; hence the two parts must be separately calculated for an exact result.

Many earthwork tables have been calculated from these formulæ, of which perhaps the best known are Bidder's (published by Vacher & Sons). Full instructions for use are printed in the tables.

Branch or Cross Roads.—In levelling for roads, railways, etc., if the proposed centre line should cross an existing work, levels should be taken at the point where the centre lines meet, and for some little distance along the existing line on each side, if there be any appreciable slope in it.

Thus in the cross roads at 13·3 chains (see the section on p. 107, fig. 81) levels are taken for 1 or 2 chains each way. These cannot be shown on the main section, but the readings for them are shown in the field book, p. 108. There are many different methods of entering the distances in such cases; in that shown, the distance *along the cross roads* is given in the distance column, and the remarks column is filled in as shown. Any clear method may be used.

When the new line is constructed, the two will cross one another, either by a level crossing or one over the other by a bridge, and this must, of course, be borne in mind when choosing the proposed formation line. We must either allow room for the bridge, or prepare for the level crossing.

In this case, we will assume that both lines are carriage roads, and that there is to be a level crossing; and that the cross road is to be raised at 13·3 chains, to bring it up to the same level as the proposed road. Then in fig. 87, taking X as the centre of cross roads, we plot up the reduced level of that point, the same as on the main section, namely 8·62. Then set off the measured distances left and

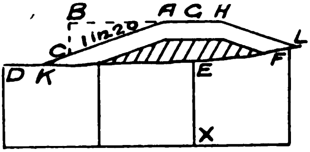


FIG. 87.

right, namely 1·00 and 2·00 to left, and 1·00 to right (see the level book, p. 108) and then plot up the reduced levels at those points. In this way we obtain the section DEF, giving the existing levels along the cross road.

Now set up XG equal to the *finished surface* level at 13·3 (that is, 11·04 + 1·75 = 12·79), and set off GA and GH, each equal to half the *finished width of the proposed road.* Then the road DEF is to be raised to cross over at AGH. It is usual to fix some gradient, say 1 in 20, to which it is to be brought, and from A and H we must then draw lines at the chosen gradient. In doing this, we must allow, of course, for the exaggeration of the vertical scale.

Thus to draw a gradient of 1 in 20 from A, we must take any two lengths, AB, BC, in the ratio of 20 to 1, but AB must be set off to the horizontal scale, and BC to the vertical scale. In the figure, AB = 66 feet to horizontal scale, and BC = $\frac{66}{20}$ = 3·3 feet to vertical scale. This fixes the finished surface KAHL of the cross road after being raised.

If now we draw a line parallel to this and 1·75 feet lower, this will show the formation line, and the shaded area shows the earth filling.

Determining Gradients.—When a line is drawn, as in fig. 81, *not* at a pre-arranged gradient, it is necessary to find what the gradient is. For this purpose we must find by measurement or calculation the formation levels (or *invert levels* in the case of sewers, etc.) at two points of known distance apart.

Say the levels are L_1 and L_2 (L_1 being the higher), and the distance D, all in *feet*.

Then the gradient is 1 in $\frac{D}{L_1 - L_2}$.

Thus for CB (fig. 81) gradient $= 1$ in $\frac{9 \cdot 36 \times 66}{8 \cdot 92}$

$= 1$ in **69·3** as marked on the figure.

Cross Sections.—Even if there are no cross roads, cross sections will be necessary wherever the ground has a transverse slope, as this slope will affect the volume of earthwork, and may affect the design.

They are—in the opinion of the writer—best taken at the same time as the levels for the main section. Thus suppose we are levelling along AB, with a down slope to the left as shown by the spot levels on fig. 88. Then we would set up the level

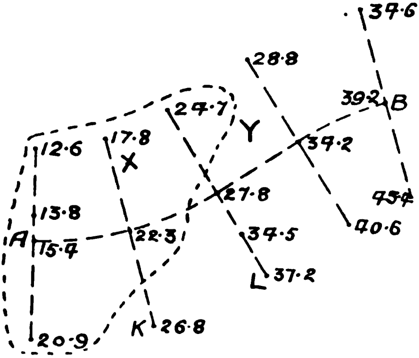


FIG. 88.

say at X, so that the lowest of the required points can be read, three or four cross sections ahead would be pegged out, and all the low levels would be read, as far as possible, from the position X, say all those enclosed within the dotted line. Then a turning-point would be chosen, to give a very low reading, as the level is to be raised for the higher points. The level would then be raised, say to Y, any points on the high ground on the back sections (such as K and L) which were invisible from X would be read, and a new lot of sections laid out, and so on.

It will be seen that if the transverse slope is great it is seldom that all the points on any cross section can be read from one position of the instrument. This is quite immaterial, however, and all that is necessary is to read the required points (whether on the main or cross sections) in the most convenient order, and to note where each point is, so that the draughtsman may plot each level in its proper position.

In pegging out the cross sections, a bundle of white pegs 18 inches or 2 feet long are a great help. Flower sticks used for tying up plants in pots can be bought for about 8d. per hundred, and answer all right where the pegs are not required to be left permanently in position. The position of each peg can be written on it in pencil (e.g. 30 *links to left at* 20 *chains*) or each peg may be numbered, while the surveyor keeps a plan on which the position of each peg is shown with its number attached.

If the distances are not too great, the numbers may be called out by the staff-holder; otherwise a good staff-holder may be trusted to book the number or position of each peg as he takes it, and the surveyor can copy them into his book *each time they meet* so as to see that the number of bookings agrees.

Booking Cross Sections.—In perhaps the most common method of booking for cross-section levels, the distance column is divided into three, headed

Left, *Centre* and *Right* respectively. Thus if a reading be taken at 30 links to the left of AB at the point where the chainage is 20 chains, we would enter 20·0 in the *centre* column, and 0·30 in the *left* column.

If the level book be not ruled in this way, any clear system can be used. The writer personally books them as in the level book below, which gives readings for the spot levels shown in fig. 88, but he does not contend that it is necessarily the best method.

It has the advantage that the position of each point is written in the same way as one would naturally think of it or describe it. Hence the writer thinks that there is less liability to error in booking when working on this system.

LEVEL BOOK FOR SOME OF THE LEVELS IN FIG. 88.

Back-sight.	Intermediate.	Fore-sight.	Rise.	Fall.	Reduced Level.	Distance.	Remarks.
6·50					20·00	—	B.M.
	13·90			7·4	12·6	1·00	Left at 0.
	12·70		1·2		13·8	·30	Left at 0.
	11·10		1·6		15·4	0·00	On centre line.
	5·70		5·4		20·9	1·00	Right at 0.
	4·30		1·4		22·3	1·00	On centre line.
	8·80			4·5	17·8	1·00	Left at 1·00.
	1·90		6·9		24·7	1·00	Left at 2·00.
13·86		·36	1·54		26·24	—	T.P.
	13·30		·56		26·8	1·00	Right at 1·00.
	2·90		10·4		37·2	1·00	Right at 2·00.
	5·60			2·7	34·5	·50	Right at 2·00.

and so on.

Other Methods.—Cross sections are taken by several other methods, however. Working on the above system, levels are, of course, taken on each cross section wherever there is sufficient change of slope to justify it. But if the transverse slope is fairly uniform on each side, a quick method is to measure the angle of slope on each side with a clinometer, the surveyor himself standing on the centre line at the point where the cross section is required.

The angle is measured in the manner described on p. 35, and entered in the remarks column. In other cases a *hand level* is used. This instrument is like a clinometer with the angle of slope permanently fixed at zero. Illustrations can be seen in the makers' catalogues. The surveyor simply holds it in his hand, standing on the centre line, brings it level, and reads the staff held at known distances along the cross section. If the rise or fall is very great, it may be necessary for the surveyor to take a reading, say at half the total required distance, then move to that point and take another reading.

In either of these methods the levels along the centre line are supposed to be taken by ordinary levelling, either at the same time as the cross sections or previously.

If the hand level is used as described, the height of the observer's eye above the ground must be measured with the staff. It will be the same each time, and serves as a backsight on the centre line.

Plotting the Cross Sections.—Referring to fig. 88 (p. 119) suppose that a road is to be constructed, having its formation level at A on the existing ground level, and rising 1 in 20 along the centre line.

A longitudinal section would be plotted, and the depths of cutting found as already described. Thus rise per chain $= \frac{66}{20} = 3{\cdot}3$ feet. Hence the formation levels at the end of each chain will be, at 0, 15·4; at 1·0, 18·7; at 2·0, 22·0; at 3·0, 25·3; and at 4·0, 28·6.

Then to plot the cross section at 0·0, in fig. 89, starting from X set off the distances 1·00 and ·30 to the left and 1·00 to the right and set up the reduced levels, 12·6, 13·8, and 20·9 at those points as given in the level book (p. 121), also at X set up the reduced level at 0·0 on the centre line, namely 15·4.

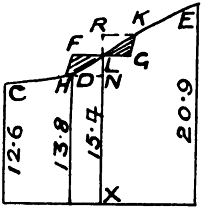


Fig. 89.

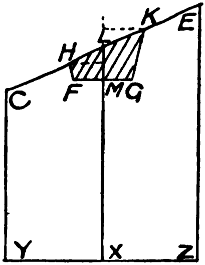


Fig. 90.

We thus obtain the line CDLE, which gives the cross section of the ground at the point A in fig. 88.

Now set up at X the formation level at 0, which in this case is also 15·4, and draw FLG, at this level equal to the formation width. Then from F and G draw the side slopes at the proper angles. Thus on the left the road will be in embankment, so that the slope is 2 to 1 (see p. 112), while on the right the slope is 1½ to 1, as the work is in cutting. In drawing these slopes the exaggeration of the vertical scale must be taken into account as already explained (p. 118). The points H and K so found will give the toe of the bank and the top or "outcrop" of the cutting respectively, and the shaded areas represent the cross-sectional areas of cutting and embankment at the point A.

Similarly in fig. 90, which represents the cross section at 1·00, we have XY = XZ = 1 chain. YC = 17·8; ZE = 26·8, XL = 22·3; XM = 18·7 the formation level at 1·00 (see p. 123), FG = the formation width, and FH, GK, the side slopes. The road is here in cutting on both sides, and the shaded area represents the sectional area of the cutting. The other cross sections are drawn in the same way. The datum line YXZ is often omitted in the finished cross section.

Volume of Cutting.—In this case the areas are usually determined graphically from the plotted cross sections, as it is not easy to calculate them from the depths on the centre line. We may divide them into triangles, or use any of the methods of chapter I. for areas. The results are then tabulated, using Simpson's rule or the trapezoidal rule.

In this case, as there are five cross sections (that is, an *odd* number), we can use Simpson's rule, and the following are the approximate results, the formation width being taken as 40 feet :—

Chainage.	Area.	Do. × Factor.
0·0	23·8	23·8
1·0	161·7	646·8
2·0	369·6	739·2
3·0	516·1	2064·4
4·0	653·4	653·4
		4127·6

The factor for the first and last areas is *unity;* for the second one it is *four*, then *two*, then *four* and so on, *four* and *two* alternately.

The last column is summed and multiplied by $\frac{66}{3}$ as the sections are 66 feet apart, and areas are in square feet.

$\therefore$ volume of cutting = 4127·6 × 22 = 90807 c. feet.

Plan of Work.—It remains to show on the plan the finished work. For this purpose in fig. 91 the line AB is reproduced without the spot levels. Now draw lines parallel to AB, and 20 feet on each side of it to represent the formation width. Thus FG = 40. Then measure on fig. 89 the horizontal distances, HN and KR, from the centre line to the end of the slope, left and right respectively, and set these off at AH and AK respectively in the plan (*note:* these distances are not to scale in the figure). The horizontal distances of H and K from the centre line are set off in the same way, left and right respectively, at the end of each cross section, giving the completed plan, without allowing for fences.

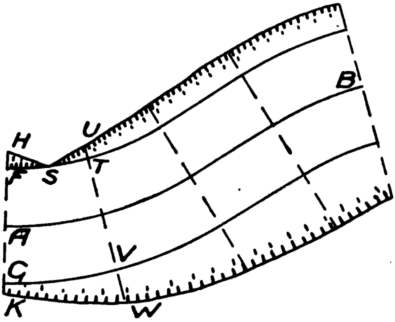


FIG. 91.

The part FH is in embankment. At some point S, between F and T, the embankment will end and the cutting will begin. There will be no side slope there, and this point S may generally be found with sufficient accuracy by producing the outer line which marks the outcrop of the cutting, to meet the line FT at S. Otherwise, we may find S by proportion thus: as the depth of embankment at F (fig. 89) is to the depth of cutting at F (fig. 90) so is FS to ST.

The area required in plan may then be found by Simpson's rule or otherwise, and an allowance made for fences if required.

To find the volume of the embankment between F and S, we take the sectional area of embankment, FHDL, from fig. 89, and multiply *half* this area by the length FS.

In this case FS = 0·5 chain = 33 feet.
Sectional area = 20 sq. feet.
$\therefore$ volume = $\frac{1}{2} \times 20 \times 33$ = about 330 c. feet.

Plan when Ground is Level Across.—If the ground is level across, the road will always be in *either* cutting *or* embankment on *both* sides, according to the longitudinal section. In this case, as we have seen, it is not necessary to actually draw the cross sections to determine the volumes, and the plan can also be found very simply from the heights. Thus in fig. 92, if the side slopes are m to 1, then

$$KL : KM :: m : 1;$$

in other words, the horizontal breadth of slope in plan = $m \times$ depth of earthwork.

Hence in fig. 91, if the ground were level across, we would set off, at 1·00 for instance, TU and VW, each equal to 1·5 × depth of cutting at 1·0 chains, the slope being 1·5 to 1.

Contouring.—The meaning of a contour line has already been defined (p. 93). To *draw* contours the usual method is to obtain a number of spot levels over the ground, and to interpolate the contours between them.

There are several methods of taking the spot levels, but when they are done by ordinary levelling they are generally taken along a series of more or less parallel lines, in the same way as for cross sections. In the case of a hill, the levels may be taken along a series of lines radiating from the summit.

The levels on fig. 88, for instance, could easily be used for drawing the contours.

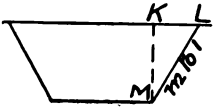


Fig. 92.

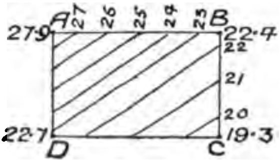


Fig. 93.

Interpolation of Contours.—In the opinion of the writer, much the most satisfactory method of interpolating the contours is *by eye.*

Thus suppose A, B, C, D, are four points whose levels are given on the diagram (fig. 93), and suppose we require to draw the contours at every single foot. Then as the level of B is 22·4, and that of A is 27·9, it is evident that between them we require to find the points whose levels are 23, 24, 25, 26, and 27.

This is done on the assumption that the slope is approximately uniform between A and B. It is assumed that, had this not been the case, extra levels would have been taken to show the positions of marked changes of slope.

Now 27·9 – 22·4 = 5·5 feet = total rise from B to A.

Hence each foot of rise will correspond with a horizontal distance of between one-fifth and one-sixth of AB. Now from 22·4 to 23 = 0·6 foot rise. Hence the distance B . 23 : BA : : 0·6 : 5·5, or B . 23 = about one-tenth of BA.

Guess the position of 23, to satisfy this proportion. Then guess the position of 27, so that A . 27 may represent a fall of 0·9 foot (from 27·9 to 27) on the same scale as B . 23 represents a rise of 0·6. Then from 23 to 27 will represent a rise of 4 feet, and we divide it into four equal parts by eye, each of which will represent 1 foot rise.

Then compare the lengths of these parts with A . 27 at the one end, and B . 23 at the other; they should each be a little longer, as judged by eye, than A . 27, and not quite twice as long as B . 23. If they do not satisfy this test then we must increase or decrease the lengths B . 23 and A . 27, and again divide up the interval. This being carried out between each pair of adjacent points, and all points at the same level joined, we obtain the contours as shown. The contours are smoothed off as much as possible when being drawn; changes of slope are usually not absolutely abrupt, and points of division should be slightly altered accordingly. Thus the distance B . 23 would be slightly increased, and B . 22 slightly diminished as compared with the values they would have had on a geometrical system of division, to give a somewhat gradual change from the close contours above to the wider ones below.

The natural tendency of the practised eye is to allow for this in division.

Interpolation by Construction.—Suppose A, B, C, are three points whose levels are given. Then taking AB as, say, 20 feet above datum, we plot a section, making A*a* = 27·6 − 20·0 = 7·6 feet; B*b* = 11·8; and C*c* = 4·7, to any convenient vertical

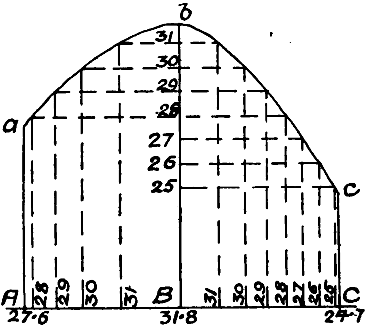


FIG. 94.

scale, the bigger the better. Then along any convenient line, say B*b*, set off whole feet to the same scale, beginning with 5 feet (*note :* the lowest level is 24·7, hence the lowest contour point will be 25), and through the points so obtained we draw horizontals as shown to meet the section line *abc*. Project the points of intersection down to the line ABC.

This method would answer well if the heights could be set off, and the parallels drawn, without error; but, practically, very great care is necessary to avoid errors which can easily be detected by eye, unless the scale adopted is inconveniently large.

Geometrical Division.—In another method, a line AB (fig. 95) is drawn on tracing paper, and divided into a number of equal parts, greater by about five than the greatest rise between any two adjacent points, while AB itself should be so long that any one division is not appreciably shorter than the estimated longest distance between contours.

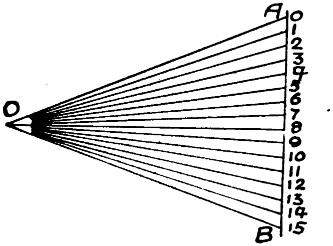


Fig. 95.

The points of division are numbered as shown, and joined to any chosen point O (OA preferably not shorter than AB) by fine carefully ruled lines.

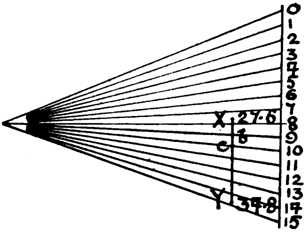


Fig. 96.

Then, to divide up a line XY, given the level at X (= 27·6) and at Y (= 34·8) we place the tracing

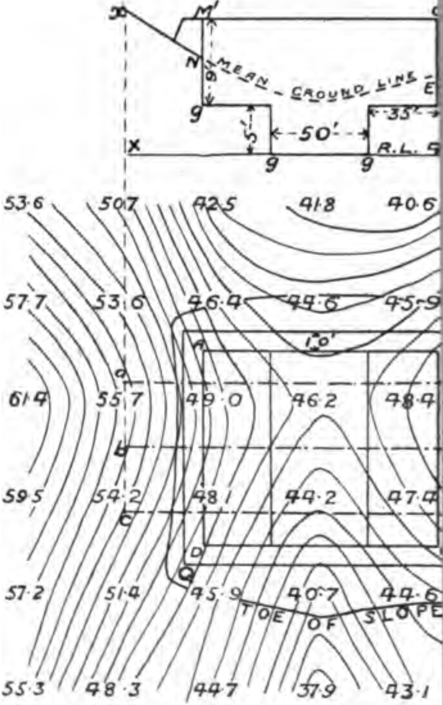


FIG. 97.

SURVEYING. —

paper on the drawing, and *keeping* AB *parallel to* XY (by eye is sufficient unless they are too far apart), we move the tracing until X comes between the 7 and 8 rays (at a point corresponding with 7·6 as well as we can judge) and, at the same time, the point Y comes between the 14 and 15 rays, namely at a point corresponding with 14·8 (*note:* 34·8 − 20 = 14·8; and 27·6 − 20 = 7·6).

When the tracing is thus fixed in position, prick through the points *b*, *c*, etc., on to the line XY.

The same tracing can, of course, be used again and again, and the method has some recommendation, but it leads, if strictly carried out, to an unnatural stiffness in the division.

In fig. 97 is shown an example of a somewhat irregular piece of ground contoured from spot levels.

Uses of Contours.—To illustrate some of the uses of contour lines in engineering drawings, on fig. 97 it has been assumed that within the area ABCD a covered service reservoir is to be built, over-all dimensions 100 feet by 50 feet, by 14 feet deep, together with one or more filter beds on each side 35 feet wide, 100 feet long over all, and 9 feet deep, outside dimensions. The bottom of the reservoir foundations to be at the level of 40·0, the top level with the top of the filters.

The earth excavated for the foundations to be made into a bank all round the work, top of bank level with top of walls, top width 10 feet, side slopes 2 horizontal to 1 vertical.

It is required (*a*) to find the volume of excavation for the foundations.

The method shown is that of "mean sections". If any section of the work be taken in a direction parallel to *aa*, the outside section of the filter beds and reservoir will be the same, and is represented by G*gg*M in the figure.

Now divide the breadth AD or BC into any number of equal parts—in the figure it has been divided into *three* parts—and *bisect* each of these parts. Through the points of bisection draw lines *aa*, *bb*, and *cc* ; then we may draw a section of the ground along each of these lines, and sketch in the *mean section* by eye, or, taking any line such as *abc* at right angles to the direction of the sections, we may read off, from the contours the levels at each of the points a, b, c, take the mean arithmetically, and plot it up in the section.

Thus taking the line *abc* on the *left* of the figure, we have

level at a = 55·4
,, ,, b = 55·2
,, ,, c = 53·8
3)164·4
mean = 54·8

Then taking XY in the section as 40, we make Xx = 54·8 − 40 = 14·8 feet on the same scale as the depths of reservoir and filters are set off.

Similarly for the points on the *right* :—

level of a = 51·6
,, ,, b = 50·9
,, ,, c = 49·3
3)151·8
mean level = 50·6

Hence Yy = 10·6 feet.

In the figure the vertical scale is exaggerated, but usually in drawings of this kind the vertical and horizontal scales are the same.

Repeating this for a number of perpendicular lines, we obtain the *mean ground line* shown, which gives approximately the mean section of the ground over the breadth AB.

If now we planimeter, or otherwise obtain, the area of the figure N$gg$N between this line and the bottom (allowing for the proper scales) and multiply by the breadth AD, we obtain the required volume.

To allow for the scales, find the area in square inches. Then if the vertical scale be 1 inch = n feet, and the horizontal scale 1 inch = p feet, then each square inch of area = $n \times p$ square feet.

Other methods of finding the volume of excavation will be given later.

Volume of Bank.—(*b*) To find the volume of the embankment round the reservoir, make GH = 10 feet, the top width of bank, and draw HR at the proper slope.

It is evident that if the area GNRH be taken, and multiplied by the length BC, the result will give the volume of the bank down the right-hand side. If the area GNRH be added to the corresponding area on the left, and the *sum* multiplied by the length BC, the result will give the volume of bank on the right and left. If now a similar *mean ground line* be drawn in the opposite direction (that is by dividing up AB or CD) the top of the walls and the mean section of bank can be shown on it, and the sum of these areas multiplied by AB or CD will give the volume of bank along the top and bottom sides of the work. This omits the four corners.

These can be dealt with in various ways. The practice of the writer is to multiply the areas by the lengths MP and PQ instead of BC and CD. A little thought will show that this includes some portion of the corners *twice*, but still omits the corners of the slopes. These errors may be regarded as balancing one another sufficiently nearly as a rule.

Determining Toe of Slope.—(*c*) To find the intersection of the bank with the existing ground in plan.

For this purpose make GH in the section = 10 feet, and draw HR at the slope of 2 horizontal to 1 vertical as before (allowing for the scales as on p. 117).

$$\begin{aligned}\text{Then the bottom level of reservoir} &= 40{\cdot}0\\ \text{depth}\quad .\quad . &= \underline{14{\cdot}0}\\ \therefore\ \text{top level}\quad .\quad . &= 54{\cdot}0\end{aligned}$$

Now set down GE equal to, say, 7 feet, on the vertical scale. Then the level of E will be 54·0 − 7·0 = 47·0. From E draw EF to meet the slope line HF, and from F project down to meet the 47 contour at the point J. Then J is one point on the toe of the slope. The same line meets the 47 contour again at S, but this point is outside the limits of the figure and is ignored.

Similarly the point L is found by projecting down from T, which is at the level of 48·0. All points so found are joined.

It is more convenient to work directly on the plan, however.

Taking the 48 contour, for instance, its vertical depth below top of bank is 6 feet. Hence, on a 2 to 1 slope, the horizontal distance corresponding to this is 12 feet. Therefore to find the point on the 48·0 contour, we simply require to find the point L whose perpendicular distance from MP is 12 feet to scale. This is done by merely sliding the scale along, keeping it perpendicular to MP by eye, and keeping zero on MP, until the 48 contour is intersected by the 12 foot division, and then marking the point L.

Similarly for J, on the 47·0 contour, the distance of J from MP must be 2 × (54 − 47) = 14 feet.

Again, assuming that the corner is a cone with P as apex, to find the point U, on the 46·0 contour, the distance PU will be 2 × (54 − 46) = 16 feet, and we take P as centre and 16 feet as radius, and cut the 46 contour at U.

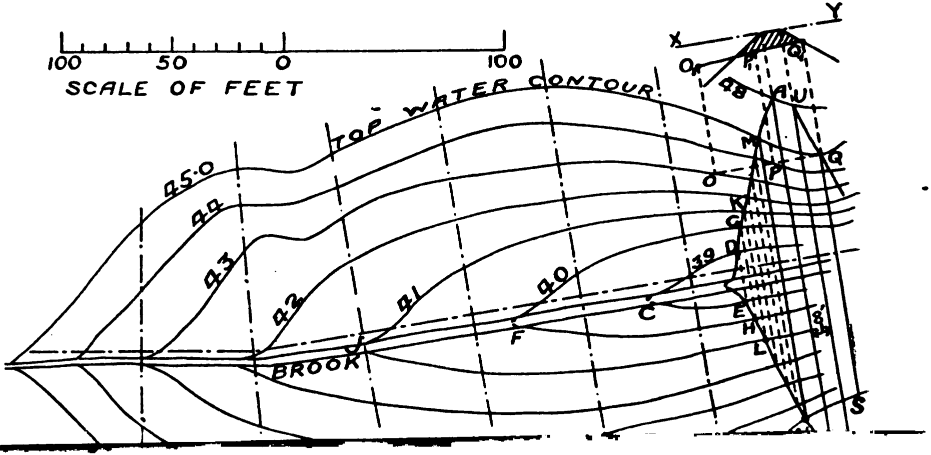
Excavations for Buildings.—The problem of finding the amount of excavation to be done for an ordinary building on irregular or sloping ground is, perhaps, most neatly solved by the method of mean sections. It is exactly similar to the example already given, except that there will generally be no bank round the outside. Another method is to actually draw several sections; find the area of excavation from each section, and then proceed by Simpson's or the trapezoidal rule.

Volume of Water in Reservoir.— Suppose that fig. 98 shows the contours on a reservoir site. The line AB represents the plan of the upstream edge of the top of the earth dam, and the upstream slope of the dam is 3 horizontal to 1 vertical. Top level of dam, 48·0; top water level, 45·0. It is required to find the volume of water in the reservoir when full to the level of 45·0.

First draw the contours MN, KL, etc., on the upstream slope of the dam, supposing the latter built. Thus for the 45·0 line, MN, we have :—

$$\begin{array}{lr} \text{level of top of dam} = & 48{\cdot}0 \\ \text{level of line MN} = & 45{\cdot}0 \\ \hline \text{vertical interval} = & 3{\cdot}0 \end{array}$$

at a slope of 3 to 1, the horizontal equivalent will be 3 × 3 = 9 feet.


SCALE OF FEET
100
50
0
100
TOP WATER CONTOUR
45.0
44
43
42
41
40
39
48
BROOK
X
Y
O
P
Q
A
U
M
K
G
D
C
E
H
L
F
J
S
8'

Hence the line MN must be 9 feet to scale from AB, at which the slope begins and the level is 48. For each line after that the additional vertical interval is 1 foot, and the additional horizontal equivalent is therefore 3 feet. In other words, the lines are 3 feet apart.

The points M, K, G, D, etc., where these lines meet the corresponding original contours give the toe of the slope, this being really the same as the second construction described for J and L in fig. 97, except that the contours on the slope were not actually required to be drawn in that case.

Now in fig. 98, the contours on the slope *close* the areas given by each of the original contours. Thus CDE represents the area of ground which will be under water, when the water stands at the level of 39·0.

FGH shows the area submerged at the level of 40·0; JKL, at 41·0; and so on.

Now suppose we take out the areas CDE and FGH, by planimeter or otherwise, reduce to square feet, and take the mean of the two. Then multiply this mean area by the vertical interval between contours (which in this case is 1 foot) and we obtain the volume of the horizontal slice (as it were) of water between the levels of 39·0 and 40·0. This process is repeated for each pair of contours, and the whole tabulated as follows .—

Level.	Area.	Ditto (Sq. Feet).	Mean.	Volume.	Total Volume (C. Feet).	Corr. Level.
39·0	·09	670				
			1930	1930	1930	40·0
40·0	·42	3190				
			6070	6070	8000	41·0
41·0	1·16	8950				
			13,230	13,230	21,230	42·0
42·0	2·28	17,510				
			22,385	22,385	43,615	43·0
43·0	3·54	27,260				
			31,765	31,765	75,380	44·0
44·0	4·71	36,270				
			40,305	40,305	115,685	45·0
45·0	5·76	44,340		115,685		

The first area is CDE, the second FGH, and so on. If the areas are found in square feet straight off, column No. 3 is, of course, unnecessary.

Column No. 4 gives the means between each successive pair of areas, and column No. 5 gives the result of multiplying these means by the corresponding vertical interval in each case.

This column might in this case have been omitted, as the vertical interval is always 1 foot.

The first line in this column gives the volume of water between the levels of 39·0 and 40·0, and hence the total volume of water up to the level of 40·0, as any water below 39·0 is negligible in amount. It is therefore repeated in the "total volume" column, and the corresponding level (last column) is entered as 40·0.

The second figure in column No. 5 gives the volume between 40 and 41, and hence, if added to the previous total volume, will give the new total volume, up to the level of 41·0, and so on.

The addition is checked by adding up the *volume* column as shown. The example, of course, shows a very small reservoir only. For larger reservoirs the interval between contours may be more than one foot.

The volume could also be found by plotting cross sections, say along the dot and dash lines (along which the original levels are supposed to have been taken). The method in this case is the same as for a road or railway cutting.

Much has been written about the relative accuracy of the two methods of working, and many authorities state that the method by contours is very much more reliable than that by cross sections.

The writer considers that the matter would still be open to argument if *only* the total volume were required. But it is evident that the table given above enables us not only to state the total volume with fair accuracy, but also to estimate fairly well the volume of water in the reservoir *when the water stands at any given level.* The results in the last two columns can be plotted in a curve for this purpose if desired.

Obviously this information may be of very great value, and this fact alone is sufficient to decide any argument as to which method should be adopted, without any consideration of relative accuracy.

Volume of Dam.—To find the volume of the dam we would require to know the top width and the down stream slope; say 8 feet, and 2½ to 1.

The contours would also have to be continued for one section farther down stream, and the contours on the down stream face filled in as before, as well as the down stream toe of slope.

We could then proceed: (*a*) by drawing a number of cross sections of the dam at equal distances, finding their areas, and using Simpson's rule or the trapezoidal rule for volume; (*b*) by the method of contours.

In the first method, referring to fig. 98, the total length of the dam is measured as 156 feet, and this may be divided into 6 equal parts of 26 feet. Let OPQ be a section plane 26 feet from one end. The section of the dam is first drawn as shown, the side slopes being drawn at the proper angles. In this case the vertical scale was taken as double the horizontal, which is shown on the drawing.

Then the line OPQ is produced to meet the 43, 44, and 45 contours at O, P, Q. These points are projected up, and taking the level of XY as 48·0, we set down 5, 4, and 3 feet respectively from it, thus obtaining the line $O_1P_1Q_1$. This line meets the side slopes, and the shaded area represents the cross sectional area of the dam along the line OPQ. The other cross sections are obtained in the same way by putting in a new ground line (instead of $O_1P_1Q_1$) for each section.

In the second method, by contours, AUTB gives the area of the bank at level 48·0; NMQS gives it at the level of 45·0; and if the mean of these areas be multiplied by the vertical interval, 3·0, we get the volume between those levels, and so on.

Laying out given Surface Gradient.—Contours may also be used to lay out on a plan the direction of a line which will be everywhere at a definite gradient without cutting or filling.

Thus suppose fig. 99 represents the contours near the top of a hill to a scale of 4 chains to 1 inch, and that it is desired to lay out on the plan a surface line starting from A and reaching the summit with a uniform slope of 1 in 20. Now we will suppose that the vertical interval between contours is 5 feet. And at a slope of 1 in 20 the corresponding horizontal equivalent is 20 × 5 = 100 feet. That is, in rising from contour to contour we must travel, in plan, 100 feet, and we must find the length, in inches on paper, which represents this.

Now 4 chains = 264 feet.

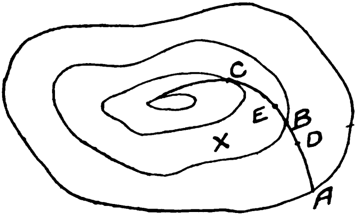


Fig. 99.

Hence to a scale of 4 chains to 1 inch, 100 feet will be $\frac{100}{264}$ = ·38 inch. Then taking this length as radius and A as centre, we cut the next contour at B. Then with B as centre, and the same radius, cut the next contour at C, and so on.

Laying Out Line on Ground.—A similar line may be laid out on the ground directly with the level. In this case the points are usually fixed at one chain apart. Thus suppose the level is set up at X, and the reading obtained on a staff at A is 13·16. Then, if a 66 foot chain be used, for a gradient of 1 in 20 the rise in 66 feet will be 66 ÷ 20 = 3·3 feet, and this will be the same for each chain length. We can then work the level book backwards.

Back-sight.	Inter-mediate.	Fore-sight.	Rise.	Distance.	Remarks.
13·16				0	Pt. A.
	9·86		3·30	1·00	
	6·56		3·30	2·00	
	3·26		3·30	3·00	
13·72		·15	3·11	—	T.P.
	13·53		·19	4·00	

Here, starting from the backsight 13·16 at A, we subtract the rise, of 3·30, to get the next reading which will be 9·86, at 1·0 chain.

Now the follower holds one end of the chain at A, and the leader takes the other end up the hill, guessing the right direction. The staff is held at the end of the chain, and the surveyor from the instrument at X notes whether the reading is more or less than 9·86. He accordingly signals the staff-holder to move *up* or *down* the hill, while the leader also swings the chain round (so enabling the staff to be kept 1 chain from A) until the correct reading is obtained, to the nearest tenth say. This fixes the first point D. Then the follower moves to D, and the reading of 6·56 must be obtained at E, and so on until we come down to 3·26 at 3 chains. We cannot subtract another 3·30, hence we take a turning-point as shown, anywhere where the reading is low. This gives a rise of 3·11. The new backsight is 13·72, say. The additional rise required to 4 chains is 3·30 − 3·11 = ·19 foot. Hence the next reading will be 13·72 − ·19 = 13·53, and so on. The points D, E, etc., are pegged out, and afterwards surveyed if desired.

If a very accurate gradient is required, the hypotenusal allowance must be made. For a rise of 3·3 feet per chain, the hypotenusal allowance is 1¼ links per chain, and the staff should be held that distance in front of the end of the chain each time.

The student should prepare tables of hypotenusal allowances like that on p. 34, for pasting into his field books. Columns should be added for rise per chain, and rise per 100 feet.

To find these, rise per chain = 66 × sin a, and rise per 100 feet = 100 sin a, where a is the angle of slope and the rise is always in feet.

Laying out Contours on Field.—Contour lines are sometimes laid out on the field in the same way, except that in this case there will be several points with the same reading, and when we move from one contour to another the readings will differ, as a rule, by a round number of feet.

This method is usually more accurate (but also much slower) than the method of interpolation.

Permanent Adjustments of Level.—It has been stated that the readings with a level are taken along a *line* or *plane* of collimation, which should be truly horizontal when the spirit level is central, and that the level is brought central in two directions, and should then remain so in all directions.

To satisfy these conditions, it is obviously essential: (1) that the plane or line of collimation should be parallel to the spirit level; (2) that the spirit level should be perpendicular to the vertical axis, round which the level rotates.

These adjustments should be accurately carried out by the makers before the instrument is sent out; but this is not always done, and, in any case, levels *will* get out of adjustment from time to time. Hence the surveyor should know how to test his instrument, and to correct it if found very defective.

These adjustments are called "permanent" adjustments (because they *should* remain correct when once corrected) to distinguish them from the "temporary" adjustments which must be carried out every time the level is used, such as bringing the bubble central, and focussing.

To understand the first test, let L be the spirit level, and ACE the line of collimation. If the latter is not parallel to the level, it will not be horizontal when the latter is central, but will make a constant angle α with the true horizontal CD, equal to the angle it makes with the spirit level (*note:* the angle is much exaggerated in the figure).

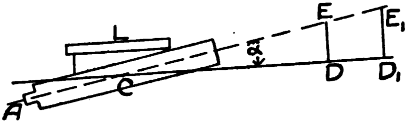


FIG. 100.

Now when we read, we are supposed to be reading along a horizontal plane or line, CD. But we will really be reading along the line ACE. Hence the *error* in the reading at any point D will be represented by DE. Its value is CD . tan α, and it will obviously be *the same in amount* for any two readings at *equal distances* from the level. For unequal distances it will be greater, the greater the distance.

Hence to test for this error we choose two convenient spots A and B, say 2 to 3 chains apart, on firm ground, and at *equal distances* from a third point C. Set up the level at C, and read the staff at A and B, correcting the bubble by the plate screws for each reading if necessary, and carefully focussing to eliminate parallax. Then as AC = AB, both readings will be increased or decreased *by the same amount*, if there is any error. Hence the *difference* between the readings will be the same as if they were both correct, and will give the *true* rise or fall from A to B.

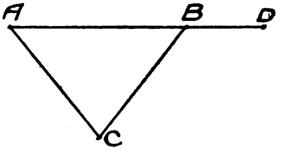


FIG. 101.

Now set up at D, in the line AB produced, say about ½ to 1 chain from B, and read the staff again at A and B, the bubble being central.

Then the distance from A to D will be three or four times as great as DB; hence the error at A, if any, will be three or four times as great as that at B, and the difference between the readings will no longer give the true rise or fall.

Let A_1, B_1, be the readings at A and B from C.
A_2, B_2, ,, ,, ,, ,, ,, D.

Then $A_1 - B_1$ should be equal to $A_2 - B_2$.

If not, there is an error.

To adjust it we may move either the bubble or the line of collimation. The former would generally be adjusted, unless there is reason to believe that the error is in the line of collimation, as, for instance, if the instrument has just been re-webbed. First suppose the bubble is to be corrected.

Work the *plate screws* a little at a time, so as to move the bubble a few divisions one way or the other, and read again at A and B. If the difference is now more nearly equal to $A_1 - B_1$ the bubble has been moved in the right direction, and we keep on moving the bubble a little more each time, and reading at A and B, till the difference becomes equal to $A_1 - B_1$.

If, after moving the bubble, the difference becomes less nearly equal to $A_1 - B_1$, we have moved the bubble the wrong way. Work the plate screws in the opposite direction, and repeat as above.

When the correct difference is obtained, the line of collimation is horizontal, but the bubble will be no longer central. Bring it back to the central position by the screws (usually capstan headed) which support it at one or both ends.

In doing this, it will be found that there are two or more screws which lock one against the other. One of these must be slackened before the other can be tightened, and the student must be careful to *finish by tightening* up the screw which was first slackened, so as to leave them both quite tight.

At the end the staff should be read again, so as to see that the reading has not altered, as it may do so if much pressure is applied to either end of the telescope in adjusting. If it *has* altered, bring it back with plate screws, and adjust bubble again.

Next suppose the line of collimation is to be adjusted.

Then after reading from D and finding that $A_2 - B_2$ is not equal to $A_1 - B_1$, work the diaphragm screws (D in figs. 72 and 73) a little at a time, and keep on reading the staff at A and B, till the correct difference is obtained.

The same remarks apply here as in the adjustment of the spirit level, and throughout the adjustment the bubble should remain central. If it moves, bring it back central by the plate screws before proceeding.

Second Adjustment.—The second adjustment is to make the spirit level (and line of collimation) perpendicular to the vertical axis.

To understand the adjustment, suppose that the level L is brought central by the plate screws, but that the vertical axis is not at right angles to it. Then the latter will not be vertical, but will be in a direction such as AB (the amount of error being, of course, exaggerated). If now we rotate the upper part of the instrument through 180° round AB, the line of collimation will take up the position shown at C_1D_1, in which the angle $AOC_1 = AOC = 90^\circ + a$, if a be the angle which AB makes with the true vertical VO. Hence $VOC_1 = 90^\circ + 2a$, or the bubble will make an angle of $2a$ with the horizontal.

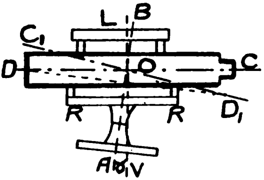


Fig. 102.

Hence to test the adjustment, place the telescope over two plate screws, and level up. Then turn through 90°, and level again. Then *back to the original position* (not through 180° from it) and level again. It will then be level in two directions.

Now turn it through 180° from the last position, and, if the bubble moves, correct *half* the movement by the screws under the telescope (in the position R in fig. 102) and the other half by the plate screws. Repeat till the bubble remains central. In working the screws R, and indeed in all permanent adjustments of the level or theodolite, the same remarks apply as in the first adjustment.

Use of Small Level.—There is generally a small level at right angles to the large one, near the object glass end. In levelling up, either for this test, or in ordinary use, the work may be somewhat shortened by setting the long level parallel to two opposite screws (supposing there are four); then level the small level *first*, with the other pair of screws; then the long one, without turning the instrument at all. Finally turn the instrument through 90°, and level with the long level.

To test the adjustment of the small level, we may proceed independently, working in the same way as described for the main level; or we may first adjust the main level so as to remain central in all positions, and then, if the small bubble is not central, bring it so by the screws which support it.

Equalization of Back- and Foresights.—It is to be understood that it is a matter of great difficulty to get the level so accurately adjusted that no error at all will remain, and it is useless to suppose that, once adjusted, the instrument will remain in perfect adjustment indefinitely. Hence, while it should be tested and corrected from time to time, it is desirable, when an accurate result is required, to work on some plan which tends to eliminate the instrumental errors, rather than torture the instrument by perpetual attempts at absolute adjustment.

Now we have seen that if the line of collimation is not truly parallel to the spirit level, this will cause *equal errors* in the readings at two points *at equal distances* from the level, and hence the rise or fall from one point to the other will be correctly obtained. Thus if all the readings could be taken at the same distance, the error would be of no consequence. This cannot well be done, but we *can* arrange that for each position of the level the *back*- and *fore*-sight shall be taken at about equal distances.

This is called "equalization of back- and fore-sights," and attention to it ensures that the error due to faulty adjustment of the line of collimation is eliminated at each turning-point, and hence is not carried forward or accumulated. There will be some residual error at each intermediate, but in this, as in all other surveying work, it is the *accumulated error* that is to be guarded against, it being comparatively easy to adjust so nearly that the error in *one* reading is of no great consequence.

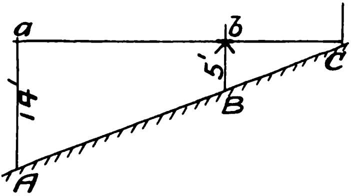


FIG. 103.

To show the importance of this, suppose we are levelling up a long hill of uniform slope, with a line of collimation sloping *upwards*, so that all readings are too *high*. Also assume that the staff is 14 feet long, and the level when set up is about 5 feet above the ground.

Also suppose that A is a turning-point (fig. 103), and that, in choosing the next position B, we choose it so as to get a backsight reading very near to the top of the staff, and that for the foresight, the staff is taken to C, so that we can just see the bottom. Then it is evident that the distance, AB, at which the backsight is taken will correspond with a rise of 9 feet (14 – 5 = 9), while the distance BC will correspond with a rise of only 5 feet, about. Hence AB will be nearly twice as great as BC, and the error in the backsight will be correspondingly greater than in the foresight.

In the case we have supposed both will be *increased* (but the backsight more than the foresight), hence the calculated rise each time we move the level will be *too great.* Hence the level at the top of the hill will be too great.

But in levelling back to our starting-point, to check, the process is reversed. The foresight will now be relatively too great each time, and we shall obtain too big a *fall* each time.

Hence we may check back correctly at the bottom, in spite of the error at the top, so that the latter would escape detection.

To assist in ensuring equalization in such cases, in the Ordnance Survey and elsewhere, 10 foot staves have been used, the idea being that they are about twice the height of the level.

But if a short piece of levelling, involving few positions of the instrument, is to be done with an instrument in fair adjustment, the use of the 14 foot staff may save time without introducing any very serious error, and it frequently enables an intermediate in a hollow to be read, which would otherwise necessitate another shift of the instrument.

Elimination of Second Error.—If the level is not perpendicular to the vertical axis, the bubble will not remain truly central in all positions. In this case the bubble should be exactly corrected with the plate screws for each back- and foresight, after aiming at the staff. Many makers supply small mirrors which fit on to the bubble tube, so that the position can be seen from the eyepiece end of the telescope by slightly raising the eye.

Reciprocal Levelling.—Now suppose we wish to carry a line of levels across a wide river or ravine

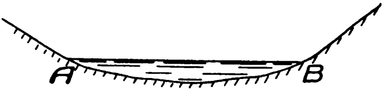


FIG. 104.

AB, say 500 feet wide. With the level set up at A, the foresight must be taken across to B. But it may be impossible to take a backsight at anything like the same distance on the left bank. In this case we resort to reciprocal levelling, as follows: having levelled down to A, equalizing back- and foresights, we shall know the level of A; set up the level exactly over that point; read its height with the staff = A_1; then send the staff to B, and read it, = B_1.

Then suppose there is an error in the line of collimation so that readings at a distance AB are too great by an amount e. Then the true reading at B should be $B_1 - e$, and the true rise from A to B is $A_1 - (B_1 - e) = A_1 - B_1 + e$.

Now take the same level and set it up at B; read its height above the ground, (= B_2), and send the staff to A and read it, (= A_2).

Then the true reading at A should be $A_2 - e$, as the distance is the same as before.

$$\begin{aligned} \text{Hence true fall from B to A} &= A_2 - e - B_2. \\ \therefore \text{ True rise from A to B} &= A_2 - e - B_2. \\ \text{But true rise from A to B} &= A_1 + e - B_1. \\ \therefore\ 2 \times \text{rise A to B} &= (A_1 + A_2) - (B_1 + B_2) \\ \text{or true rise A to B} &= \tfrac{1}{2}\{(A_1 + A_2) - (B_1 + B_2)\}. \end{aligned}$$

This eliminates the unknown error e.

To eliminate irregularities of refraction, two levels are sometimes employed for a very accurate result. Two staves should be held on A and B, and the levels set up touching them, as nearly as possible. The height of each level is read directly, and simultaneous readings are then taken through each level to the opposite staff. The levels are then reversed, the one from A being taken to B, and vice versa, and the readings repeated.

If the distance is too great for the staff graduations to be read directly, the staff-holder is directed to use a pointer, which he runs up or down the staff, till it coincides with the hair, when the observer at the level signals, and the staff-holder reads the position of the pointer.

Curvature and Refraction.—It has been stated that levels are given above some level surface or datum. For very small areas such a surface may be regarded as a plane, but for bigger areas a level surface is everywhere parallel to an imaginary surface passing through the mean level of all large seas and oceans, and as it were produced through all land surfaces to meet the corresponding mean level of the next ocean. Hence all such surfaces are nearly spherical in shape, and may, in fact, be regarded as portions of a sphere whose radius is equal to the mean radius of the earth.

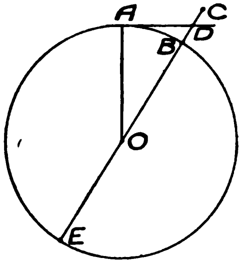


FIG. 105.

Thus A and B, fig. 105, are on the same level. But if a level be set up at A and levelled, its line of collimation will be in the direction AD, and the point D will appear to be on the same level as A. Thus the rise from A to C, as given by the reading with the level would appear to be DC, whereas the true rise is BC.

The apparent rise is thus too *small* by the amount BD, which is called the *curvature correction.* To find it, by Euclid :—

$$AD^2 = BD \, . \, DE. \quad \therefore \; BD = \frac{AD^2}{DE}.$$

Now BE is the diameter of the earth, and BD is much greater in proportion than it could ever be in practice. Without sensible error we may take DE = BE.

$$\therefore \; BD = \frac{AD^2}{BE} = \frac{(\text{distance})^2}{\text{diameter of earth}}.$$

If the distance AD of the reading be one mile, and the Earth's diameter be taken as 8000 miles, then BD = $\frac{1}{8000}$ mile = $\frac{63360}{8000}$ inches, or 8 inches nearly.

Thus the curvature correction is about 8 inches for a distance of 1 mile, and for other distances is proportional to the square of the distance.

For $\frac{1}{4}$ mile distance it is $\frac{8}{16} = \frac{1}{2}$ inch nearly.

Equalization of back- and foresights eliminates this error as well as that due to faulty adjustment. The method of reciprocal levelling also has the same effect. Curvature correction must be *added* to a *rise* and *subtracted* from a *fall*, reckoned from the instrument. It has, in fact, the effect of making the staff reading too big.

Refraction.—Now in consequence of the fact that the density of the atmosphere decreases with the height above sea-level, the path of a ray of light through it is not straight, but becomes curved by atmospheric refraction, as shown in fig. 106. Thus the rays of light from C appear to be coming from C_1.

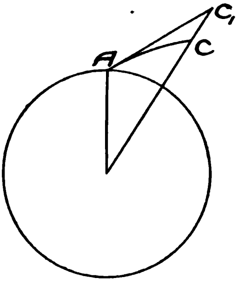


FIG. 106.

The methods by which the amount of the refraction correction is arrived at are beyond the scope of this book, but it is sufficient to say that, on the average, it appears to be about *one-seventh* of the curvature correction, and in the opposite direction. Hence the total correction is about *six-sevenths* of the curvature correction calculated as above.

Degree of Accuracy in Levelling.—The only criterion of any value, in levelling, as in any other kind of surveying, as to the degree of accuracy to be aimed at, is that the total amount of error should not be so great as to be of importance according to the work in hand, and the writer rather deprecates the use of fixed standards. But as a standard to set before students he adopts an error of 0·01 foot for a 400 foot reading, and for other distances an accumulated error of $\sqrt{n} \times \cdot 01$ where n is the total number of back- and foresights. This would give 0·04 foot per mile, which can be obtained with care, and represents pretty good work.

Contours by Hand Level.—Some reference has already been made to the hand level (see p. 122). It is used in several different ways, but its special advantage is its portability, and quickness of adjustment. It is, of course, much less accurate than a dumpy level, and could not be used for a long line of levels if a good result were required. For short lines such as cross-sections, however, where there is less chance of accumulated error, it may save time, especially if the levels along the centre line have been previously taken.

Most frequently the observer measures the height of his eye above the ground, and standing directly on the point of known level, reads the staff on the required point, or directs the man up or down hill, to obtain the required reading, if the contour is being laid out on the field.

With the hand level it is usual to complete each cross-section before passing on to the next, the observer always moving on to the point where the foresight was taken, and using his own height as the new backsight.

The writer prefers the hand level fitted with the Grubb sight (see p. 80).

Exercises for Revision.

1. Draw up an imaginary field book for the spot levels shown in fig. 97, showing the whole fully made up and checked. As

sume a staff 14 feet long, and that the first reading is on a B.M. at the level of 60·0.

2. Taking a centre line running from west to east on a certain piece of ground, the following table shows the spot levels :—

Reduced Level.	Distance.			Reduced Level.	Distance.		
	Left.	Centre.	Right.		Left.	Centre.	Right.
81·1	150	0	—	27·2	—	150	75
66·9	75	0	—	14·3	—	150	150
51·7	—	0	—	9·5	—	225	150
36·2	—	0	75	22·8	—	225	75
23·7	—	0	150	38·4	—	225	—
19·5	—	75	150	52·1	75	225	—
33·3	—	75	75	67·0	150	225	—
46·7	—	75	—	66·5	150	300	—
62·2	75	75	—	49·0	75	300	—
74·2	150	75	—	33·2	—	300	—
68·8	150	150	—	16·8	—	300	75
56·9	75	150	—	3·4	—	300	150
42·2	—	150	—	75·0	—	B.M.	—

All distances are in feet.

Draw up an imaginary field book for these levels, showing the whole fully made up and checked. Plot the levels and draw the contours at every 5 feet. A square 150 feet each way in the centre of this ground is to be levelled off to the level of 40·0, the side slopes to be $1\frac{1}{2}$ to 1 in cutting and 2 to 1 in embankment.

Draw the plan of the finished work; show the altered contours along the slopes, and find the volumes of cutting and embankment.

3. On the same piece of ground as in exercise No. 2, starting from the south-east corner (reduced level = 3·4) it is desired to run a path 10 feet wide at an even gradient of 1 in 9·5, up to the 40 foot contour, so as to enter the central portion on the level. Trace the direction of the centre line, so as to give this gradient without cutting or filling. Then through the points so found take a smooth curve to represent the actual centre line; draw the longitudinal section; the cross sections at every

50 feet, and the finished plan. Assume that the formation width is 10 feet, and that the formation level is the same as that of the finished surface, and that the gradient along the altered centre line is uniform. Find the volumes of cutting and embankment.

4. The levels for a small reservoir are as given in the table on p. 155. Two lines AB, BC, are laid out, each 200 feet long, and making an angle of 165° with one another. The cross sections are taken at right angles to these lines, at every 50 ft. from A; the cross section at B bisects the angle ABC.

Draw the contours at every foot, and if the top water level be 45·5 feet above datum, and the top level of dam 48·0; upstream slope 3 to 1, top width 8 feet; downstream slope $2\frac{1}{2}$ to 1, and upstream edge of top at right angles to BC and 150 feet from B. Find the volume of water in the reservoir: (*a*) when the level is 41·0; (*b*) when the level is 45·0; find also (*c*) the total volume of the dam above original ground level; (*d*) the total area of ground covered by the work, from the top water contour on the upstream side to the toe of the downstream slope. If it is desired to place two outlet valves from the reservoir, of which the upper one shall be able to draw off just half the total volume from 45·0 downwards, find the level at which it should be placed.

(*N.B.*—The bed of the stream is omitted in the table and may be neglected. The lowest line of levels is on the left bank. Distances are horizontal.)

5. Draw the exercise given in fig. 97 to the scale of 20 feet to an inch. The spot levels are 50 feet apart each way. Find the volume of excavation by the method of mean sections, and find the top breadth of bank (by a process of trial and error) so that the total volume of bank may be the same as the volume of excavation. For this width draw the bank in plan.

6. What are the curvature and refraction corrections for a distance of 1400 feet? Reciprocal readings are taken between A and B, at that distance, as follows: at A, height of level = 4·7, reading to B = 7·37; at B, height of level = 5·0, reading to A = 2·23. Find the level of A if that of B be 20·27, and find the collimation error for a reading of that length, if the refraction be assumed constant. What will be the collimation error with the same level for 400 feet distance?

TABLE FOR QUESTION 4.

Reduced Level.	Distance.			Reduced Level.	Distance.		
	Left.	Centre.	Right.		Left.	Centre.	Right.
48·7	—	0	—	51·7	—	250	150
42·5	—	50	5	44·8	—	250	75
47·2	—	50	50	36·8	—	250	13
53·4	—	50	100	42·1	75	250	
46·8	50	50	—	52·8	150	250	
50·6	100	50	—	47·8	100	275	
49·8	100	100	—	49·4	150	300	
43·8	50	100	—	41·8	75	300	
40·6	—	100	8	34·7		300	9
44·5	—	100	50	38·6		300	75
50·7	—	100	100	48·2		300	150
53·6	—	150	150	49·4		350	100
44·5	—	150	75	35·6		350	50
39·8	—	150	4	32·2		350	5
42·8	75	150	—	38·6	50	350	
49·3	150	150	—	50·2	100	350	
50·1	150	200	—	51·5	150	400	
43·5	75	200	—	36·2	75	400	
38·4	—	200	1	30·7		400	2
43·4	—	200	75	37·5		400	75
51·7	—	200	150	50·7		400	150

7. Plot the example in fig. 88, taking the formation level at A the same as the reduced level, and a gradient therefrom of 1 in 20. Plot the longitudinal section, the cross sections, and the plan, and find the volumes of cutting and embankment, and the area of ground in plan. The formation width to be 40 feet.

CHAPTER IV.

TRIANGULATION SURVEYS.

Object and Procedure.—One of the principal difficulties in surveying is to obtain accurate measurements of horizontal distances, especially on uneven ground, and these errors tend to accumulate.

Hence in big surveys it is usual to cover the area with a number of isolated points or stations as shown in fig. 107, each station being visible from every other station with which it is connected by a line in the figure. Only one of the sides (which in ordinary triangulations are, say, from ½ mile to 5 or 10 miles long) need be suitable for direct measurement, but this one, say AB, must be measured with extraordinary precautions to ensure accuracy. Then all the angles of all the triangles shown are measured with very great accuracy, and, starting from the triangle ABF,

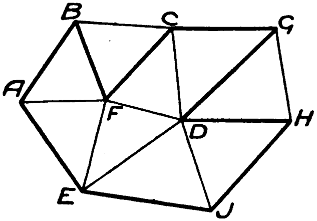


Fig. 107.

of which one side is known, all the remaining sides are calculated, each triangle, in turn; being solved from one side and the three angles. Finally, when all sides and angles are known, any closed circuit is chosen (say as shown by the thickened lines) so as to pass through all the stations and close back to the starting-point, and this figure is then treated as a traverse and the co-ordinates calculated.

When the survey is bigger, the stations are divided into districts, as a rule, so as to make a number of traverses, instead of one very big one.

The advantage of this method is that the positions of the points are fixed entirely by angular measurements, which can be taken much more accurately than is practically possible with linear measurements.

Then, starting from any station E, the details are surveyed in all directions by chain surveying, traversing, or whatever method may be desired, and, when we arrive near any other station, say J, the work is checked on to that station by measuring to it; there will nearly always be some error; but this error will not have had time (so to speak) to accumulate to a very large amount in the distance between stations, and it can then be distributed over the various small stations between, so as to make the position of J from the detailed survey agree with its position from the triangulation.

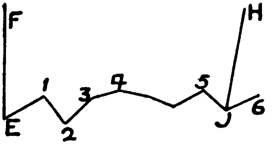


Fig. 108.

Thus in fig. 108, we might start from E, take traverse stations 1, 2, 3, etc., and, taking F as back-station, observe the included angles, FEI, E12, etc. Then get the bearing of FE from the triangulation (FE, not EF) and, having chained the distances E1, 12, etc., we start with the co-ordinates of E already known, and calculate the remaining co-ordinates to J, which should agree with those from the triangulation.

Any small error is distributed by the rules already given (*see* Traverse Surveying).

If 5 be the station before J, and we observe the angle 5 JH, we have an independent check on the angular measurements, for we can get the bearing of JH from the triangulation sheets.

Then by the rules for calculating bearings we have: Bearing of JH—Bearing of FE = sum of observed included angles—$n \times 180°$, where n is some integer. The left-hand side of this equation is known from the triangulation. We can then continue the traverse in the same way. These traverse lines can then be used as base lines for chain or other surveys.

Branch Traverses.—The student must note carefully that when a traverse branches off from a triangulation, as in this case, or from another traverse, he must be careful to connect up the new work with the old, by observing the angle between the first new line (in this case E 1) and a line of the previous or main survey, in this case EF. Other instances are given in example No. 6, p. 90.

Measurement of Base Lines.—A complete discussion of the measurement of the base line AB, so as to ensure the highest possible accuracy, is beyond the scope of this book. It must suffice to note that the chief difficulties are to follow the ever-varying length of the rod, or tape, employed (in consequence of changes in temperature, humidity, etc.) and to fix the exact point at which one rod, tape, or chain ends, and to ensure that the next shall either begin from exactly the same point, or, if not, that the distance between the end of one and the beginning of the other shall be accurately known.

For a description of the methods and artifices which have been used in the past to overcome these difficulties, and an account of the results achieved, the student is referred to larger works on surveying.

We may mention here, however, that the steel tape, used in some such way as described on p. 85, is sometimes used for small surveys.

For more extensive surveys, invar tapes are sometimes used now. This is an alloy of nickel and steel, in which it appears that with a rising temperature certain molecular changes take place, producing a contraction which very nearly balances the expansion produced by the rising temperature. Thus the temperature effect is extremely small.

For a paper and an important discussion dealing largely with this subject, the reader is referred to the "Proceedings of the Irish Institution of Civil Engineers" for the year 1906.

The various corrections to the measured base are applied as described on pp. 85 to 87.

Sometimes, however, the tape is stretched between pegs, instead of being allowed to lie along the ground. In this case an additional correction for sag must be introduced. Its value per chain is $\frac{L}{24}\left(\frac{WL}{T}\right)^2$, where L = length of chain, = distance between pegs; T =

tension on chain; W = weight per foot, both in pounds. This correction is to be subtracted.

Measurement of Angles.—The instrument most often used for measuring the angles is a transit theodolite.

The general method of using a theodolite has already been described, but there are many special precautions which have to be observed when great accuracy is required. The errors to which the angles are liable may be divided into three groups:—

(*a*) Non-instrumental errors.
(*b*) Instrumental errors.
(*c*) Accidental errors, to which no definite cause can be assigned, and which may or may not be instrumental.

Non-instrumental Errors.—Non-instrumental errors are due to: (1) Bad centring over the station; (2) observing to a high point on a signal which is not truly vertical. This can only be remedied by having the signals stayed in that position, or by having some one in charge of each during observation; (3) atmospheric refraction, which is avoided by observing on dull calm days, or with luminous signals at night; and (4) personal bias, whereby it appears that each observer, when bringing the spider lines on to a signal by means of the tangent screw, has always the tendency to either go too far, or not far enough, according to the observer's individuality. This is partially eliminated by bringing the hairs on to the signal from *left to right*, say, for the first reading, and from right to left for the second round.

Instrumental Errors.—Instrumental errors are those which arise in consequence of some fault in the instrument itself. These are:—

(1) The error due to irregular graduation of the circle. This can be overcome by the method of "repetition," already referred to (see p. 66).

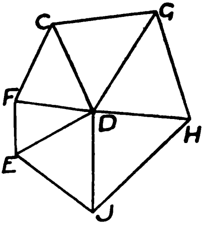


FIG. 109.

But repetition is a tedious process when applied to several angles at the same station, and it eliminates no *other* errors. Hence it is usual to read all the angles at any station, say D (fig. 109) in a continuous *round*, as it is called, starting from any one, say C, as zero, and working right round the circle back to 360° at C, taking the reading at each station, and finding the angles by repeated subtraction.

Both verniers are read at each station (there are usually *two* on the smaller theodolites but often *three* on the larger sizes), thus giving all angles on two (or three) parts of the scale.

Having completed the round, we turn the lower plate through 90° (as well as we can judge by eye) and clamp it in that position. Then turn the telescope over, revolving it in a vertical plane, round the horizontal axis, and then turn the upper plate so as to look again at the first station C, and take the reading there, and then at each other station, reading both verniers and finding the angles by repeated subtraction as before.

This process of turning the telescope over, is known as "reversing face," and the method here described of measuring the angles with "both faces," and with other special precautions, is sometimes called *reiteration*, to distinguish it from *repetition*, already described.

So far as the elimination of graduation errors goes, it will be seen that by turning the bottom plate through about 90°, and reading both verniers, each angle is obtained on four different parts of the scale. If there are three verniers, we turn the bottom plate through about 60°, and get each angle on six different parts of the scale.

(2) Errors of eccentricity.

The centre of the vertical axis, round which the upper plate and verniers revolve, should be in exact agreement with the centre of the graduated circle. But it is obviously almost impossible to secure this absolutely, and, to take an exaggerated case, let C be the centre of the graduated circle, and C_1 the centre of the vertical axis. Now suppose we start with vernier number 1 at O, and vernier number 2 at P, and turn the instrument until vernier number 1 comes to A. Then as the turning takes place round C_1, the angle described is OC_1A. Now draw B_1CB parallel to C_1A. Then $OCB = OC_1A$. But the arc OB on the graduated circle measures the angle OCB, and gives the true measure therefore of the angle OC_1A. Whereas the vernier reading gives the measure of the arc OA, and hence *too small* a value for the angle described.

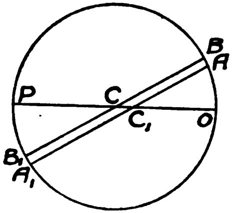


Fig. 110.

But vernier number 2 will be at A_1 when number

1 is at A. Hence the arc measured by number 2 vernier will be PA_1, whereas the true angle is measured by the arc PB_1. Hence the value given by this vernier will be *too great* by the arc B_1A_1 which is practically equal to AB.

Hence this error is eliminated by reading both verniers and taking the mean.

It can be similarly shown that the error is eliminated by taking the mean of *any number* of equidistant verniers.

(3) Errors due to faulty adjustment.

There are certain "permanent adjustments" in the theodolite, as in the level, which can be checked, and, if necessary, corrected by the surveyor from time to time without the aid of the instrument-maker. Any error in these may lead to some error in the angles, unless the instrument is used so as to eliminate them.

It is proposed to treat them separately now.

Permanent Adjustments of Theodolite.—(*a*) *To make the plate levels perpendicular to the vertical axis.*

Set one of the plate levels parallel to two opposite plate screws. The other level is usually at right angles. Clamp the outer axis (that is the lower limb or plate) and bring both bubbles central, the upper plate being unclamped. Then turn the instrument through 180°, and if either bubble moves correct *half* the movement by the screws which support that bubble, and the other half by the plate screw or screws. Repeat till perfect. If there is only *one* level, work in the same way as described for the second adjustment of the Dumpy level. The principle of the two adjustments is the same.

When both bubbles remain central in all positions, clamp the upper plate and unclamp the lower, and turn the instrument. If the bubbles move, the axes are not parallel. This is technically described by saying that the *centres are out of truth,* and it can only be remedied by the maker.

Any slight error in the adjustment of these levels is of little importance. Hence the levels are seldom very sensitive.

(*b*) *To make the line of collimation perpendicular to the trunnion axis* (*or horizontal axis*).

If fig. 111 represent a plan of the telescope when horizontal, and AB be the trunnion axis, then the line of collimation (along which we read) should be perpendicular to AB, as shown by CD. Then when the telescope is revolved round AB, the line of collimation will describe a true plane, perpendicular to AB. But if the line of collimation be in a direction such as EF, then it will describe a double cone, and if the telescope be turned over vertically for half a revolution, E and F will move to E_1 and F_1, and we will be looking along E_1F_1, whereas before we were looking along EF.

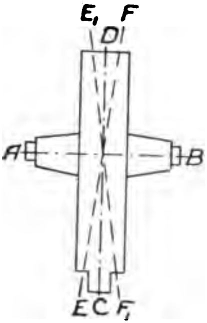


Fig. 111.

Hence to test for this error, choose a piece of level ground, say about 400 or 500 feet across, and set up the theodolite near the middle, and level it.

Let fig. 112 be a plan, and let AB be the direction of the trunnion axis, and CD the perpendicular to it, with which the line of collimation should correspond. If not, let OE be the direction of the latter, and a the angular error. $\therefore$ DOE $= a$.

Direct the instrument carefully on to any well-marked point E, with both horizontal motions clamped, and the telescope horizontal or very nearly so.

Now unclamp the vertical motion, and turn the telescope over, round AB as axis. OE_1 will now be the line of collimation. Bring the telescope about horizontal, and direct an assistant to make a mark E_1 in line with the cross hairs, the observer signalling him into position.

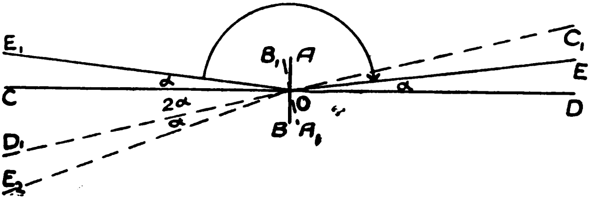


FIG. 112.

Then unclamp the upper horizontal plate, and turn the instrument in a horizontal direction, round the vertical axis, so as to bring the line of collimation back to the point E. It is evident, as shown by the arrow, that the horizontal angle described will be 180° − $2a$.

B will move to B_1, A to A_1, C to C_1 and D to D_1, so that $BOB_1 = DOD_1 = 180° - 2a$. A_1B_1 is the new position of the trunnion axis, C_1D_1 that of the perpendicular, and OE that of the line of collimation.

Now unclamp the vertical motion, and turn the telescope over again, round A_1B_1 as axis. E will move to E_2, and OE_2 will be the line of collimation. Direct the assistant to make a mark at E_2 in line with the hairs. Then if there is an error, E_2 will not coincide with E_1. Evidently $E_1OE_2 = 4a$; and $D_1OE_2 = a$. Hence we may take $E_2D_1 = \frac{1}{4} \times E_2 . E_1$. Hence we mark D_1, to satisfy this, and then turn the diaphragm screws to bring the hairs into coincidence with D_1. The line of collimation will then coincide with the perpendicular OD_1.

Any small error in this adjustment produces little or no effect on vertical angles. It is fortunate that the effect is negligible, for it cannot be eliminated by reversing face. The effect on horizontal angles between objects at different altitudes may, however, be considerable.

Thus *one minute* of collimation error, will produce errors of 1′, 2′, or 11′, in the horizontal angle between two points, one horizontal and the other at an altitude of 60°, 70°, or 85° respectively.

This error, fortunately, is eliminated by reversing face.

It is well to have a wall on which to make the marks E_1, E_2, D_1.

(*c*) *To make the horizontal or trunnion axis perpendicular to the vertical axis.*

Referring to fig. 113, which in this case is an elevation, if AB be the axis, which is not horizontal, then assuming that the line of collimation is now perpendicular to it, that line will now describe an inclined plane EF (instead of the vertical plane CD) when the telescope is revolved round AB.

Hence to test the adjustment, set up near a high building or other object, so as to be able to get an angle of elevation of about 45° to some well-marked point, say the tip of a lightning conductor, E. Level up very carefully, as if for vertical angles (see p. 171), taking care that the legs are well pressed down, and keeping the outer axis clamped all the time. Then direct on to E, fig. 114, unclamp the vertical motion, and turn the telescope down as low as possible, so as to look at points preferably not less than 20 or 30 feet away. Mark a point F in line with the hairs.

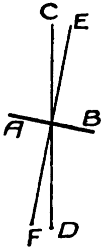


FIG. 113.

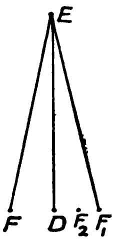


FIG. 114.

Then E and F, if there is any error, will lie in an inclined plane as shown. The figure assumes that the *left* end of the horizontal axis is too high. Now reverse face. That is, turn the telescope over vertically, and then round horizontally (unclamping the upper plate for this) so as to look back at E.

The horizontal axis is now reversed. The high end will be on the *right*, and if we turn the telescope down again it will describe the plane EF_1 sloping the opposite way. Mark the point F_1 in line with the hairs, then mark D midway between F and F_1. Then ED is the true vertical plane which the line of collimation should describe. If F_1 is to the right of F, the right end of the horizontal axis (as seen in the second position) is too high. One end of the axis is adjustable by a pair of capstan-headed screws, by means of which we raise or lower that end a little, as may be required. Then reset on E, and come down again and note the point F_2 at which we arrive. Then adjust again accordingly, and in a few trials we will move correctly from E to D.

Both screws must be tight at the end of the adjustment. See the remarks on p. 143.

A second method is by the *striding level*. This instrument is not supplied with all theodolites, but if there is one, to test this adjustment level up with care as above, and place the striding level across the horizontal axis. Call one end of the striding level A, the other B.

Then suppose we place the level on so that A is on our *right* as we face the instrument. Then read both ends of the level. The tube is graduated as shown, and the reading is taken from the centre outwards. Thus in the figure the readings would be entered as

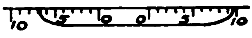


FIG. 115.

A, on right, 9·6 ; B, on left, 6·8.

Then reverse the striding level, so that A is on the *left*, and read again.

Add up the *left* readings, and add up the *right* readings, and *one-quarter the difference* will give the error in the horizontal axis.

Thus suppose we have :—

A on right 9·6, B on left 6·8
B on right 8·8, A on left 7·6
18·4 15·4
left = 15·4
4) 3·0
0·75

Hence the error is 0·75 of one division on the tube; and the *right* end is too high (as the readings there add up the higher).

Hence we must adjust the movable end of the axis to bring the *right* reading *down* by 0·75; that is, leaving the level in the last position, we must bring the right end of the bubble to 8·8 − ·75 = 8·05 and the left end to 7·6 + 0·75 = 8·35.

The proof is beyond the scope of this work. If this method is used, this adjustment may be made *before adjustment* (*b*). But if the first method be adopted, as here described, then the *order* of adjustment must be as here given.

The writer wishes to lay stress on this point, because many books are in error with regard to it, and any error in adjustment (*b*) would affect the adjustment of the horizontal axis if done by the first method.

Any error in this adjustment causes a negligible error in vertical angles, which, however, is not eliminated by reversing face. The error in the horizontal angle between two points at different altitudes may be considerable, but *is* eliminated by reversing face.

A slope of *one minute* in the horizontal axis will cause errors in the angle between two points of the same order as the collimation error.

(*d*) *To make the line of collimation parallel to the spirit level on the telescope.*

This adjustment can be made by the same method, and in the same way, as described for the Dumpy level, if the level is attached to the telescope.

In many theodolites, however, the level is on the T-piece which carries the verniers of the vertical arc.

In this case the line of collimation should be parallel to it when the vernier reads zero only, and to test the adjustment, level up, aim the telescope at any well-marked point, then bring the bubble exactly central by the "clip screw," or T-antagonizing screws, at the side, and then adjust the hairs so as to exactly bisect the point, using the tangent screw. See that the bubble is still central, and read the vertical angle. Then reverse face and repeat. If the two results are not the same, there is an error. Set the vernier to read the *mean* of the two readings, and then bring the hairs back to the mark by the *clip screw*. This throws the bubble out, and we bring it back to the centre by the screws which support it.

The mean reading is the correct angle of elevation, so that this error is eliminated by reversing face.

Frequently, instead of trying to correct the error, we take *half the difference* between the observed angles, which is the correction to be added with one face, and subtracted with the other. This correction is called the *index error*, and should be found frequently, as it is apt to vary.

Thus if the angles observed are:—

$$\begin{array}{lr} \text{face left} & 3^\circ\ 21'\ 30'' \\ \text{,, right} & 3^\circ\ \ 5'\ \ 0'' \\ \hline & 2)\underline{16'\ 30''} \end{array}$$

index error = 8′ 15″, to be *added* to all observed angles of elevation if *face right*, and subtracted if face left.

To angles of *depression*, the rules are reversed, as these angles are *minus*.

If the level is on the telescope, we may find the index error in the same way, but in this case we cannot correct the bubble for each reading, hence we must level up with great care, as if for vertical angles (see below) to start with.

Levelling for Vertical Angles.—Where vertical angles are to be read as well as horizontal (for instance in tacheometry, and usually in triangulation) the levelling must be very carefully done.

First level up with the plate levels. Then if the upper bubble is on the telescope, set the vertical vernier carefully to zero. But this is unnecessary if the level is on the T-piece.

Next look at the upper bubble, and if it is not central, bring it so by the clip-screws. (*Note:* we here assume, to start with, that the vertical axis has been brought truly vertical by the plate levels; then this last adjustment will bring the upper bubble perpendicular to it, and all will be right. But if it be not truly vertical, we have introduced two equal and opposite errors as in fig. 102, p. 144, and we will have a bubble movement equal to the sum of these errors if we turn the instrument through 180°.)

Then turn horizontally through 180°, and if the upper bubble moves, correct *half* the movement by the clip screws, and half by the plate screws. Then through 90°, and correct *all* with the plate screws. Everything should again be perfect. Turn through 180°, and correct *half* of any movement with the clip screws and half with the plate screws. Then through 90°, and all with the plate screws, and so on till perfect.

After the first station, the clip screws should acquire little adjustment, and it may save some time, when we first look at the upper bubble, to correct any error with the plate screws instead of the clip.

While the levelling is being done, the outer axis should be clamped. Then clamp the inner axis, unclamp the outer, and turn the instrument. If the bubbles move the centres are out of truth, and the instrument cannot be used for accurate measurement of vertical angles, unless we either use the level on the T-piece and correct it for each reading, or clamp the bottom plate in position before levelling.

Comparison of Theodolites.—The size of a theodolite is technically fixed by the diameter of the graduated circle. A 4 or 5-inch theodolite usually reads to 1 minute, or sometimes 30 seconds, of angle. A 6-inch reads to about 10 seconds.

There are many different methods of graduating the vertical circle. The only point upon which the writer would insist is that the system of numbering should be such that, given the face, the reading itself should tell whether the angle is elevation or depression.

For this reason the writer declines to have a theodolite which reads zero on the horizontal and up and down from that, as in Fig. 116.

Not only is it necessary to specially note which way the angle is, but the figures on the vernier must run in the same direction as those on the main scale. And as these run in opposite directions, we must have two sets of figures on the vernier, one for angles of elevation, and the other for depressions, and there is always the possibility of reading the wrong set of figures.

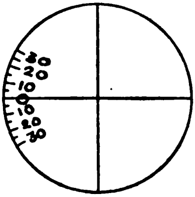


FIG. 116.

Supposing this condition satisfied, the method of finding the angle of elevation or depression for any given reading will depend upon the exact system of graduation, and upon the face used, and must be studied on the instrument.

As regards the upper level, the writer thinks it is best on the T-piece in theodolites, where vertical angles, if measured at all, are usually measured with both faces, as in triangulation or astronomical work. Because in this case the bubble can be watched all the time, while the index error cancels out. But where readings are taken with only one face, as, for instance, in ordinary tacheometry, he prefers the level on the telescope as the connexion between it and the line of collimation is then more rigid, and once the level is adjusted, it is less likely to develop a variable index error.

If there are *two* levels, use the one on the T-piece for readings with both faces, and the one on the telescope when using only one face.

Hints to Beginners.—Beginners should note that the plumb-bobs are liable to unscrew, either at the bob itself or at the top, and must be attended to. It is best to place the bob in the pocket when moving the instrument from station to station.

In wet weather water sometimes gets between the parts of the object glass. In this case it must be unscrewed for cleaning, but great care must be taken to put the parts back in the same positions; the cleaning should be done with a soft cloth, and, before unscrewing the object glass, a mark should be made to enable it to be screwed up again to the same point. Otherwise the line of collimation may be thrown out of adjustment.

Face.—A reading taken with the vertical circle on the observer's *left* as he looks through the eyepiece, is usually said to be a "face left" reading; when we turn the telescope over, the vertical circle will be on the *right*, and the reading is a "face right" one.

Final Adjustment of Angles.—When all known sources of error have been eliminated, there will still remain accidental errors in the angles. These must be detected and scientifically adjusted in accordance with mathematical principles. This adjustment is beyond the scope of this book however.

The corrected angles are then used for the final computation of sides.

The Three-Point Problem.—Given the co-ordinates of three triangulation stations, A, B, C, it is required to find the co-ordinates of an unknown point X, by observing the angles from X to A, B, and C.

This problem arises sometimes when it is desired for any reason to fix a new triangulation station at some point X, say to check up some important work there, or when it is desired to start say a tacheometric survey at a point X previously unfixed, and to connect it up with a pre-existing triangulation ABC.

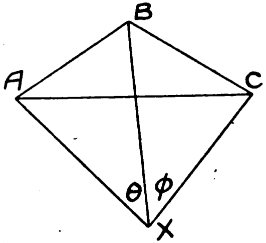


FIG. 117.

From the triangulation sheets, or from the co-ordinates, we can find the lengths and bearings of AB, BC, and CA, and the angles between them, as described in chapter II. (pp. 81 to 83).

Now suppose the angles θ and ϕ are observed (fig. 117).

Then in the triangle XBA, $\dfrac{BX}{AB} = \dfrac{\sin BAX}{\sin \theta}$

and in the triangle XBC, $\dfrac{BC}{BX} = \dfrac{\sin \phi}{\sin BCX}$.

By multiplication $\dfrac{BC}{AB} = \dfrac{\sin BAX}{\sin BCX} \cdot \dfrac{\sin \phi}{\sin \theta}$

$$\therefore \underline{\frac{\sin BAX}{\sin BCX} = \frac{BC}{AB} \cdot \frac{\sin \theta}{\sin \phi} = k, \text{ say,}}$$

where k can be calculated, as all the terms are known.

$$\text{Now } BAX + BCX + ABC + AXC = 360^\circ$$
$$\therefore \underline{BAX + BCX = 360^\circ - ABC - (\theta + \phi) = A, \text{ say.}}$$

Here A can be calculated, as ABC, θ, and ϕ, are all known.

Thus we know the ratio $\dfrac{\sin BAX}{\sin BCX} = k$

and the sum $BAX + BCX = A$.

The first step is to find each of these angles, BAX and BCX.

Now BAX = A − BCX.

$$\therefore k = \frac{\sin(A - BCX)}{\sin BCX} = \frac{\sin A . \cos BCX - \sin BCX . \cos A}{\sin BCX}$$

$$= \sin A . \cot BCX - \cos A.$$

$$\therefore \underline{\cot BCX} = \frac{k + \cos A}{\sin A} = \underline{k . \operatorname{cosec} A + \cot A.}$$

This gives BCX. Then BAX = A − BCX.

Then CAX = BAX − BAC,

and ACX = BCX − BCA.

Then solve the triangle ACX from three angles and the side AC, to find AX and CX.

Lastly, given the bearing of BA, and the angle BAX, we can find the bearing of AX, and hence the difference of latitude and departure from A to X, and the co-ordinates of X. Check by working from C to X.

If the points be so arranged that a circle will pass through A, B, C, and X, then BAX + BCX = 180°. In this case sin BAX = sin BCX, and the value of k is independent of the actual values of these angles, so that the problem becomes insoluble. In fact X might lie at any point on the circle, and the angles θ and ϕ would be the same. The more nearly the value of A approaches to 180°, the less reliance we can place on the results. Hence the points should be chosen so that a circle will not pass through them. A fairly good result is generally obtained if A, B, and C can be so chosen that X lies inside the triangle ABC.

Graphical Solution.—Graphically, set off BAD = θ, and draw AO_1 perpendicular to AD (fig. 118). Bisect AB at E, and erect a perpendicular EO_1.

The point O_1 is the centre of a circle passing through A, B, and X. Similarly find O_2 from the line BC, and the point X in which the circles meet is the required point. This construction is probably not very often used.

In hydrographic work the angles θ and ϕ are laid off on tracing paper or on an instrument called a "Station pointer" (for which see the catalogues), and this is then moved about on the drawing until the rays pass through the points A, B, C. This has the disadvantage of not exhibiting clearly the accuracy (or otherwise) of the fix, while in the construction above, if O_1 and O_2 are not close together, and the circles meet at a good angle, we know that the fix is probably good.

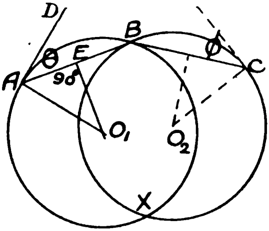


FIG. 118.

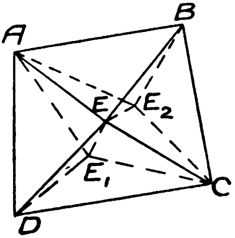


FIG. 119.

Satellite Stations.—Suppose A, B, C, D are four stations forming a polygon, and it is desired to take a central station at or about the position E. No point can be found there, from which we can see to A, B, C, and D, but at E there is, say, a church spire or a big tree on which we can fix a flag, that would be visible *from* all those points, but at which we could not conveniently set up a theodolite. At E_1

we find a point from which we can see A, C, and D as well as E; and from E_2 we can see A, B, C, and E.

Then we observe *to* E, from the surrounding stations, and take E as the true central station, but observe the central angles *from* E_1 and E_2. It is required to find from these what the values of these angles would have been if observed at E itself.

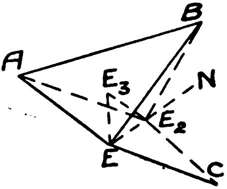


FIG. 120.

Thus in fig. 120, given AE_2B, it is required to find AEB.

$$\begin{aligned} \text{Now } AE_2N &= AEN + EAE_2 \\ \text{and } BE_2N &= BEN + EBE_2 \\ \text{by subtraction, } AE_2B &= AEB + EAE_2 - EBE_2 \\ \text{that is, } AEB &= AE_2B - EAE_2 + EBE_2. \end{aligned}$$

The angle EAE_2 and EBE_2 may, if E_1 and E_2 are fixed in time, be observed from A and B while the theodolites are there. But if not they can be fairly easily calculated.

AB will be known, at any rate approximately, from the previous work, and AE and BE can therefore be approximately found from the angles at A and B. Then E_2E must be found, either by direct measurement, or indirectly, say by measuring a base E_2E_3 and reading the angles E_2E_3E, and E_3E_2E.

$$\text{Then } \frac{EE_2}{BE} = \frac{\sin EBE_2}{\sin EE_2B}.$$

$$\therefore \sin EBE_2 = \frac{EE_2}{BE} \cdot \sin EE_2B.$$

EE_2B is directly observed, and the other quantities found as above. EAE_2 is found in the same way.

If the line EE_2 passes *inside* the triangle—as in the case of BCE—the formula is slightly modified. It is easy to show that it becomes $BEC = BE_2C - EBE_2 - ECE_2$.

Reduction of Base to Sea Level.—Referring to fig. 121, if AB be a measured base, at a height BC above mean sea level, it is desirable to reduce it to its equivalent length CD at sea level, that, this being done for all countries, surveys of adjacent countries may join together properly.

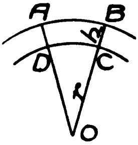


FIG. 121.

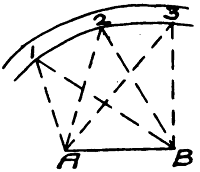


FIG. 122.

Obviously if $OC = r$ = radius of Earth
and $BC = h$ = height of base,

$$\text{then } CD = AB \,.\, \frac{r}{r+h} = AB\left(\frac{1}{1+\frac{h}{r}}\right) = AB\left(1+\frac{h}{r}\right)^{-1}$$

∴ the correction to be applied

$$= CD - AB = AB\left\{1 - \frac{h}{r} - 1\right\} = -\, AB \,.\, \frac{h}{r} \text{ very}$$

nearly.

Sketch Surveys.—A simple kind of survey, sometimes called a triangulation survey, but better described as a sketch survey, is carried out by measuring a base line AB (fig. 122) and from the ends of it, reading the angles to leading points 1, 2, 3, etc., on the ground to be surveyed.

A sketch is made on the ground showing these points with numbers attached to correspond with the numbers in the angle book. The sketch shows how all the details lie with respect to these points and to the base line. The base line is first plotted. Then the measured angles are laid off, to fix the points by intersecting rays, and the details filled in by eye from the sketch.

The angles may be measured by theodolite, or by the prismatic compass, in which case we read the bearing of AB both at A and at B, and also the bearings of all other points.

EXERCISES FOR REVISION.

1. A steel tape is 100 feet long with a pull of 18 lb., at a temperature of 62° F. Sectional area ·01 sq. in.

It is used to measure a base with a pull of 25 lb. at a temperature of 70°. The tape is stretched between pegs at the ends, so as to hang clear of the ground, and the rise for each chain, from peg to peg, is as follows, viz., + 0·82; + 1·21; + 1·28; + 1·06; + 0·24; – 0·45; – 0·48; – 0·89; – 1·25; + ·12; + 1·24 feet.

A *minus* result means a fall. For the last fraction of a chain the rise is 0·55.

The tape weighs ·03 lb. per foot length, and the mean of three measurements gives 1148·24 feet.

Find the corrections for (*a*) slope; (*b*) tension; (*c*) temperature; (*d*) sag; also the corrected length of the base line.

2. Find the index error of a transit theodolite, if the angles of elevation as observed to a fixed point are:

$$\text{face left, } 5^\circ\ 17'\ 10''$$
$$\text{face right, } 5^\circ\ 21'\ 40'',$$

and find the corrected angles if the following angles of elevation are observed,

face left, (*a*) $+\ 2^\circ\ 12'\ 10''$; (*b*) $-\ 7^\circ\ 54'\ 20''$.

3. The readings of a striding level on a trunnion axis are :—

A on left, 10·0 ; B on right, 3·8
B on left, 6·6 ; A on right, 7·2

Find the values to which the B and A readings must be corrected, the level remaining in the last position, that the axis may be horizontal.

4. Given AB = 2127·2 feet; BC = 2987·6 feet; and angle ABC = 102° 15′; also $\theta = 91° 24'$, and $\phi = 48° 12'$, in fig. 117, find the angle BAX and BCX. Then find the angle ABX and the length BX, and calculate the co-ordinates of X if B is 5796·4 N. and 9684·2 E., and the bearing of AB is 67° 10′ E. of N.

5. Referring to fig. 120, if E_2E_3 = 200 feet, CE = 3876·8; BE = 4969·5, and the following angles are observed, namely E_3E_2E = 60° 12′; E_2E_3E = 47° 10′; EE_2B = 115° 12′; EE_2C = 251° 18′; find (*a*) the angle E_2BE; (*b*) the angle ECE_2; (*c*) the angle BEC.

Also find the angle AEB if AE = 6912·4 feet, and EE_2A = 60° 12′.

6. Distinguish between a traverse and a triangulation survey as regards (*a*) the immediate object to be achieved; (*b*) the method of field work.

7. Find the length of the base in Q. 1, reduced to mean sea level if the mean height above sea level is 586·8 feet, and the mean radius of the Earth be taken as 20,926,000 ft.

CHAPTER V.

TACHEOMETRIC SURVEYING.

General Account.—Tacheometry is a method of surveying in which the instrument is set up at a station A, and a graduated staff is held at the points, 1, 2, 3, etc. (fig. 123), which are to be shown. By reading the length of staff intercepted between two spider lines in the instrument, it is possible to find the distances A1, A2, etc., from the instrument to the staff; the horizontal angles between the rays, A1, etc., are measured at the same time with the tacheometer, each angle being usually taken from some known or standard direction.

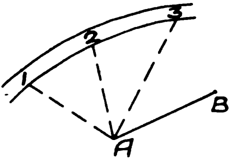


FIG. 123.

Knowing the angles and distances it is obvious that we can plot the positions of the points 1, 2, etc., and these points are then joined up according to a sketch made in the field book on the ground.

Tacheometer.—The tacheometer is similar to a theodolite in every respect, except in the arrangement of the spider lines in the diaphragm.

The usual arrangement consists of five horizontal lines as shown in fig. 124, and one vertical line. Only the three hairs *a*, *b*, *c*, will be considered for the moment, and it is to be understood that, in use, we look at a staff, like a levelling staff, held on the point which we desire to fix, and read where it is intersected by each of the hairs *a*, *b*, *c*.

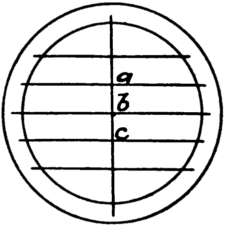


Fig. 124.

Principle of the Method.—Referring to fig. 125, let ABC be the staff, O the object glass, and F the principal focus of the object glass, outside the telescope. Also let *a*, *b*, *c* be the cross-sectional view of the hairs. Then when the staff is focussed, the image of A is at *a*, and that of B at *b*.

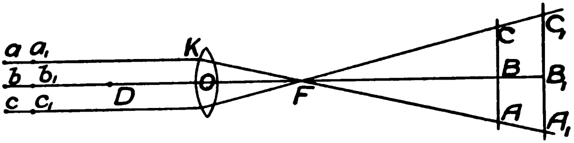


Fig. 125.

Now of all the rays which go to form the image, consider those which, inside the telescope, are parallel to the axis O*b*. Outside the telescope these rays must have passed through F, and must there have made a constant angle with the axis OB, whatever the focus; for in altering the focus, we simply move the hairs *a*, *c*, parallel to *b*O, as shown at a_1, c_1, and hence do not alter the direction of the rays considered.

Therefore, wherever the staff may be, we have by similar triangles,

$$\frac{\mathrm{FB}}{\mathrm{AB}} = \frac{\mathrm{FO}}{\mathrm{OK}} = \frac{\mathrm{FO}}{ab},$$

neglecting the thickness of the lens.

$$\therefore \mathrm{FB} = \mathrm{AB} \,.\, \frac{\mathrm{FO}}{ab}.$$

But $\mathrm{AC} = 2\mathrm{AB}$; and $ac = 2ab$, if the hairs are equidistant.

Hence $\mathrm{FB} = \mathrm{AC} \,.\, \frac{\mathrm{FO}}{ac}$.

Now FB is the distance of the staff from the focus F of the object glass; AC is the length of staff intercepted between the hairs a and c, and is found by subtracting the reading of a from the reading of c. It is called the staff intercept, and is usually denoted by s.

FO is the focal length of the object glass, denoted by f; ac is the distance between the outer hairs, and is the length of the image of AC; it is denoted by i.

Hence $\mathrm{FB} = s \times \frac{f}{i}$. If D be the centre of the instrument, the distance DB is really required, instead of FB. But the length FD is constant. It is called the *additive constant*, and if we denote it by k, we have

$$\mathrm{DB} = s \times \frac{f}{i} + k.$$

The ratio $\frac{f}{i}$ is called the *multiplying constant*. It is the aim of the maker to so adjust the hairs that its value may be some round number, usually 100, and it is usual for the makers to inform the purchaser of this value, as well as of the value of the additive constant k.

No careful surveyor, however, will trust the word of the makers as to the value of the constants, and they should always be tested.

For this purpose find k independently as follows: focus the hairs distinctly by moving the eye-piece, and then focus the telescope on some distant object, and measure the distance from the object glass to the diaphragm screws. This gives the focal length f, and the value of FO. Then measure from the object glass to the trunnion axis. This gives OD, and the two are added to find FD, or k.

Then, on level ground, measure out distances of, say, 100, 200, and 300 feet from the centre of the instrument, and read the staff at each position, the telescope being horizontal, or nearly so, and great care being taken to see that the staff is vertical for all readings.

Let s_1, s_2, s_3, be the staff intercepts.

$$\text{Then } 100 = s_1 \times \frac{f}{i} + k.$$

$$200 = s_2 \times \frac{f}{i} + k, \text{ and so on;}$$

k, s_1, and s_2 being known, $\frac{f}{i}$ can be found from each of these equations. The various values should agree closely, and, if so, the mean is taken.

Sometimes the equations are solved in pairs, simultaneously, to find k, as well as $\frac{f}{i}$, instead of finding k directly.

In addition to this test, the same permanent adjustments require attention as in the transit theodolite, and in particular the index error for vertical angles must be known, or, preferably corrected (see pp. 170 and 171), for, as we shall see, there is much work with vertical angles in tacheometry.

Anallatic Lens.—To avoid the necessity of adding the constant k for each point, Porro of Milan inserted an extra lens, which he called an anallatic lens. This is represented by K in fig. 126, O being the object glass.

Following out the same rays as before, from a and

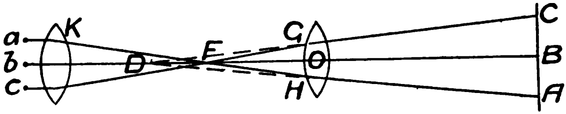


Fig. 126.

c backwards, they must pass through the focus F of the *anallatic lens*, at a constant angle. Hence (as K and O are fixed with respect to one another) they will strike the object glass O at fixed points G and H, and in fixed directions. Consequently their paths GC and HA, outside of the object glass, will be fixed also, and, if produced backwards, they will therefore meet the axis of the telescope in a fixed point D, and at a fixed angle.

These both depend upon four quantities, namely (a) the distance ac; (b) and (c) the focal lengths of the anallatic lens and object glass; (d) the distance between these lenses.

It is the object of the maker to so adjust these quantities that D may coincide with the centre of the instrument, and that the constant angle BDC may be such that its cotangent is some round number, usually 200.

$$\text{Then } \frac{\text{DB}}{\text{BC}} = 200, \text{ or } \frac{\text{DB}}{\text{AC}} = 100.$$

$$\text{Hence } \underline{\text{DB} = 100 \text{ AC}}.$$

The constant multiplier in this case is ½ cot BDC, and is no longer represented by $\frac{f}{i}$; while there is no additive constant, all distances being obtained directly from D.

The value of the multiplying constant may be found experimentally as before, omitting k from the equations on p. 185.

Although invented to avoid the additive constant, this is by no means the chief advantage of the anallatic lens.

In the opinion of the writer spider's webs are the best lines for reading by, and it will be gathered that it is exceedingly difficult to adjust the distance between them with such nicety that the constant multiplier shall be exactly 100, and even if they are specially mounted so as to be adjustable (of which the wisdom is doubtful), the very tiniest movement makes a big percentage difference. Now without the anallatic lens there is no remedy for this, and many surveyors have to use constant multipliers like, say, 98·7, necessitating charts or special tables for finding the distances.

But with an anallatic lens, we have seen that the constant multiplier depends upon the distance between the lenses. Hence the anallatic lens is mounted upon a special rack, and after proportioning all the rest as exactly as possible, a final adjustment of the distance between the lenses is made by direct experiment so as to bring the constant multiplier exactly right.

It is true that this slightly alters the position of D, but this effect is entirely negligible in comparison with the other.

This use of the anallatic lens is well understood by English makers of surveying instruments, and some of them supply special keys, so that the adjustment can be tested, and if necessary corrected, from time to time. The writer recommends the young surveyor not to buy a tacheometer unless it is so fitted.

Generally, therefore, we have
distance = $cs + k$, where c is the multiplying constant, and k is zero if there is an anallatic lens.

This assumes that, as in the figures, the staff AC is at right angles to the axis of the instrument.

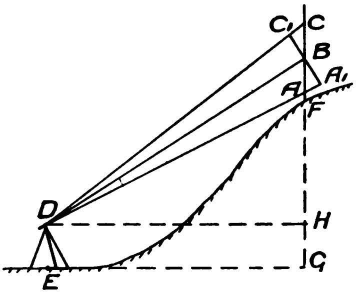


FIG. 127.

On Sloping Ground.—Now suppose the ground on which we are working slopes as shown in fig. 127. Then with a vertical staff at F, we obtain an intercept AC, or s; and, knowing the constants of the instrument, we require to find the horizontal distance EG.

In this case we require to read also the vertical angle HDB, which we will call a. This is read on the vertical arc, and corrected for index error, if any. It is evident that the instrument must be levelled up carefully, as on p. 171.

Now AC is not perpendicular to the axis DB of the instrument, and the first step is to find what the intercept AC *would* have been if the staff had been perpendicular to DB.

Draw C_1A_1 perpendicular to DB.

Then $A_1BA = 90° - HBD = HDB = a$.

And $AA_1B = DBC_1 - A_1DB = 90°$ very nearly, as A_1DB is very small.

Hence $\frac{BA_1}{BA} = \cos A_1BA = \cos a$ very nearly.

$$\therefore BA_1 = BA \,.\, \cos a$$

$$\text{or } \underline{A_1C_1 = AC \,.\, \cos a = s \,.\, \cos a.}$$

But $DB = c \times A_1C_1 + k$, by the principle of the tacheometer.

$$\therefore \underline{DB = c \,.\, s \,.\, \cos a + k}.$$

But $EG = DH = DB \,.\, \cos a$.

$$\therefore EG = c \,.\, s \,.\, \cos^2 a + k \cos a.$$

The last term is usually taken as equal to k.

Hence, $\underline{\text{horizontal distance} = c \,.\, s \,.\, \cos^2 a + k}$

where k is zero if there is an anallatic lens.

We can at the same time find the rise or fall from E to F.

For $HB = DB \,.\, \sin a$.

$$= c \,.\, s \,.\, \cos a \,.\, \sin a + k \sin a.$$

The last term is sometimes omitted, whether there is an anallatic lens or not, as it is always small.

But rise = GF = HB + GH − FB.

Now GH is the height of the instrument above the ground, and FB is the reading on the staff of the middle hair *b*.

Hence, substituting for HB, we have:—

Rise = $c \,.\, s \,.\, \cos \alpha \,.\, \sin \alpha$ + *ht of tacheometer*
— *reading of middle hair.*

if α be negative (that is, an angle of depression) the first term is negative.

The three terms are summed algebraically, and the result *if plus* means a *rise*, but *if minus*, *a fall*, from the station E to the point F.

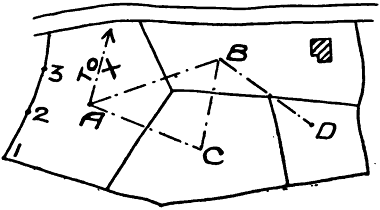


Fig. 128.

Thus in the field book, we require columns for the height of the tacheometer, the reading of the middle hair, and the vertical angle, as well as for the two outer hairs and for the horizontal angle from some known point as zero, the latter being required for plotting.

General Procedure.—Thus if fig. 128 represent a piece of ground to be surveyed, we choose a station A to begin with.

If the survey is to be connected up with a triangulation, A may be fixed by the three-point problem, or may be itself a triangulation station. We observe from it to any other triangulation station as zero, say a station X. Then send the staff to the points 1, 2, etc., and at each point read the horizontal angle, the vertical angle, and the readings of the three hairs, and read the height of the instrument itself, by placing the staff beside it.

From these the distances, as well as the rises and falls, are calculated, and the reduced levels worked out, for which purpose the level of A must be known.

New stations, B and C, are chosen to carry on the survey, and readings are taken to them from A. It is a good plan to take the readings between stations with both faces, noting in the remarks column which are face left and which face right.

Unless there are permanent signals up, say at triangulation stations which are visible, it is wise at each station to set the horizontal vernier to zero, and to direct, by unclamping the lower plate, to some well-marked point, which may or may not be shown on the drawing.

Then, having taken all the readings required from that station, we go back to this mark, and see that the vernier comes back to zero.

This ensures that there has been no movement of the bottom plate, and saves the time which would otherwise be necessary in sending the staff-holder a second time to the back station.

Thus suppose we move to B.

We level up for vertical angles, set the vernier to zero, choose say the tip of a withered branch of a tree, and set to zero on it. The staff-holder, meanwhile, has been going back to A. We then take two complete sets of readings to A, one face left, and one face right. Then reverse face again to bring the instrument back to that face which is ordinarily used (generally *left*), take all required points as usual (except that whenever reading to a station as C, or D, we read with both faces), and finally check back to the withered branch.

In the column headed "station and zero" we write the name of the station, and, under it, a description of the point taken as zero, so as to avoid any bad effects from a lapse of memory.

In plotting, we take no notice at all of the withered branch.

The plotting may be done by scale and protractor, entirely, or the stations may be plotted by co-ordinates like an ordinary traverse.

Suppose the readings to ordinary points are taken with *left* face.

Then, at station B, say, add or subtract 180° from the *face right* readings to A and D (*adding* if under 180°, and *subtracting* if over); the result should nearly agree with the face left angle, and we then take the mean in each case. Then mean reading at D − mean reading at A = clockwise angle ABD. If the reading at D is less than at A, add 360° to it before subtracting.

This gives the included angle. The distances AB and BD are each found from the mean of four sets of tacheometric readings, namely face left and face right, from A to B, and from B to A, and so on. The co-ordinates can then be calculated as usual.

The points are usually plotted with scale and protractor, after the stations are all plotted. For this purpose, set the protractor with its centre on B, and so that the mean reading at A, as found above, comes on the line BA. Weight it down in that position, and mark all the horizontal angles with needle points, numbering them in soft pencil, to correspond with the numbers in the book. Then remove the protractor, place the scale so that its edge passes through each point in turn (zero being at B) and scale off the horizontal distance. Erase the old number, and write the number again close to the actual point. If levels are also required, write down the reduced level of the point at once.

When several points have been plotted, join up according to the sketch (p. 182), and interpolate the contours, if desired, as the work proceeds.

Some surveyors prefer to direct to magnetic north as zero, by the compass, instead of to objects such as withered branches. It has the advantage of giving a rough direct check on all bearings, but the writer thinks it less sensitive as a means of detecting any slight movement of the bottom plate, such as that caused by accidentally turning the wrong tangent screw.

If possible the stations should be arranged for observations in circuits. In some cases it may be possible to arrange them all in one big circuit like a closed traverse. But more often it is easier to make a large number of small closed circuits, as, for instance, by observing, in fig. 128, from A to B and C; from B to A and C; and from C to A and B.

Then, so far as levels go, the algebraic sum of the rises and falls, A to B, B to C, and C to A, as given by the means of the four readings in each case, should be zero. If nearly so, but not quite, any small error is distributed over the rises and falls, in the same way as described for latitudes and departures in traversing, to make them close, before calculating the reduced levels of the stations.

If co-ordinates are calculated, ABC is treated as an ordinary closed traverse, the included angles being found as already described.

If plotted graphically, suppose we start at A, fig. 129, and plotting round the figure we come back to A_1 instead of A. Then AA_1 is called the *closing error*. It should not be great, but the positions of B and C should be adjusted to correct it.

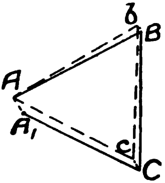


FIG. 129.

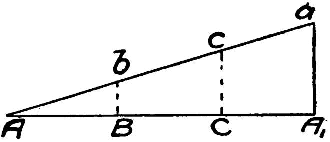


FIG. 130.

For this purpose, from B and C draw Bb and Cc parallel to A_1A, and in the same sense.

Then mark off Bb, the length of which is found from the proportion

$$Bb : A_1A :: AB : \text{whole perimeter}$$

$$\text{and } Cc : A_1A :: AB + BC : \text{whole perimeter.}$$

The lengths of each side being known, the whole perimeter is found by addition.

Bb and Cc may be found graphically as in fig. 130. Set off the lengths of the lines (to reduced scale if desired) on a horizontal $ABCA_1$, as shown. Make A_1a = closing error. Then Bb and Cc will give the corrections at B and C.

Often we take the sides as equal, then, for three sides, B*b* = *one-third* of A_1A; C*c* = *two-thirds*, and so on.

It is to be understood that the same process of graphical correction can be applied to an ordinary traverse.

When, but not before, the positions of the stations are corrected, we proceed to plot the isolated points.

Similarly the reduced levels of the points are not worked out till the levels of the stations have been adjusted.

The Field Book.—Many forms of field books have been used from time to time. That arranged by the writer has fifteen columns headed as follows:—

(1) station and zero; (2) point; (3) horizontal angle; (4) readings; (5) generating number; (6) angle from vertical; (7) angle of elevation; (8) difference of level; (9) instrument and middle readings; (10) difference; (11) rise; (12) fall; (13) reduced level; (14) distance; and (15) remarks.

The filling in of column no. (1) has already been referred to. No. (2) gives the number attached to each point or station.

No. (3) gives the horizontal angle as read, always on the same vernier; and no. (4) gives the readings of the *two outer hairs* (*a* and *c* of fig. 124).

No. (5), the generating number, is the difference between the readings multiplied by the constant multiplier; that is the value of cs in the formulæ.

No. (6) is inserted because many tacheometers read the angle from the zenith instead of from the horizontal. This column is unnecessary if the instrumental reading gives the angle of elevation directly. If the angle *is* from the vertical, then we subtract the figure in column (6) *from* 90° to get the angle of elevation, which is, therefore, *plus* or *minus*, according to whether the figure in column (6) is under or over 90°. This is filled in in column no. (7) with the proper sign attached, and gives the value of α in the formulæ.

The "difference of level," column no. (8), gives the value of $s \cdot c \cdot \cos \alpha \cdot \sin \alpha$, and is either worked out by ordinary logarithms, or obtained from tables or by slide rule. It is entered *plus* or *minus*, according to the sign of α.

In column no. (9) we enter the height of the instrument in the first line. This line contains nothing else, except the name (or number) of station in column (1) and the reduced level of the station in column (13).

Then in each line after the first in column (9), we fill in the *reading of the middle hair* at the point to which that line refers.

Column (10), "difference," means the result of *plus* the height of the instrument *minus* the middle reading. That is, subtract the middle reading *from* the height of instrument as given in the first line. The result is plus or minus accordingly.

Then combine algebraically the results of columns (8) and (10), and the result *if plus* is a *rise*, and *if minus*, *a fall*, *from* the station to the point. This if a rise is *added* (and if a fall subtracted) to the reduced level of the station, which is written in the top line of column no. (13), to give the reduced level of the point.

The distance (col. (14)) is the value of $c \,.\, s \,.\, \cos^2 \alpha$; which like col. no. (8) is either calculated in the ordinary way or obtained from special tables.

Thus whenever we move to a new station the first line is filled in only for columns no. (1), no. (9) and no. (13). The readings to points begin in the second line.

The field book should be interleaved with blank pages for the sketches.

The numbering of points may begin from zero at each station, but it is better to go on up to, say, 100, before beginning again, so that no two points of the same number will be likely to be near together. Otherwise confusion may arise.

The field books as recommended may be purchased from the author (price 2s.) either bound for triangulation, astronomical, tacheometric and traverse surveys combined, or for any one of these kinds of survey separately. Full description of filling in accompanies each book.

Tacheometric Tables.—There are several tacheometric tables in the market. The most convenient known to the writer are Jordan's, which can be obtained from Messrs. T. Cooke and Sons, Great Chapel Street, London. The description is in German, and the generating numbers go up to 250 only, as the tables are really intended for a metric staff, and 250 metres is a long reading. But it is easy to divide the generating number by two or three, and then multiply the figure from the tables by the same factor.

Messrs. Kern of Aarau make a special slide rule for giving the values of $N \cos^2 a$ and $N . \sin a . \cos a$, which is supplied with their plane tables, etc.

Plotting Scale.—A special apparatus for plotting has been used. It consists of a kind of combined scale and protractor. A description of such an apparatus will be found in Proceedings Inst. C.E., 1913, and a similar apparatus has been made by Mr. J. H. Steward, Strand, London.

Degree of Accuracy.—The degree of accuracy obtainable in tacheometry depends largely upon the instrument, and upon the care taken in determining the values of the constants, etc., and still more upon the weather.

From some experiments made by Mr. R. E. Middleton at Wimbledon it appears that an accuracy of about 1 part in 2000 was reached, for distances.

But these results can hardly be expected in ordinary work, and probably the surveyor will find that with an ordinary 5-inch tacheometer he cannot get much beyond 1 in 500.

Check on Readings.—If the hairs are equally spaced, the reading of the middle hair should be the mean between the other two readings. This gives a useful check, and all readings should be tested by this rule before signalling the staff-holder to move.

Extreme Hairs.—The two extreme hairs (see fig. 124) are generally intended to have a multiplying constant half as great as the inner ones.

They may be used for short readings, whereby the accuracy is doubled if we read to the nearest hundredth only, or they are sometimes useful if the inner ones get broken.

The eyepiece is generally movable to enable it to be brought directly opposite each of these hairs for convenience in reading. If, on moving the eyepiece, the reading appears to alter, there is some parallax, which should be corrected (see p. 105).

EXERCISES FOR REVISION.

1. It was desired to test a tacheometer at a place where no level ground was available. The focal length of the object glass was 11 inches, and the distance from the object glass to the trunnion axis was 6 inches. There was no anallatic lens.

Distances of 100, 200, and 300 feet were chained out on ground sloping 5° upwards from the instrument, and the staff intercepts were 1·00, 2·01, and 3·02 feet. Find the additive and multiplying constants.

Also if the reduced level of a station A be 327·9, and if the tacheometer be 4·8 feet above the ground, and the readings in the table (p. 199) be obtained to two points 1 and 2, find the reduced levels of these points, and plot them relatively to A and one another.

Then find the direction of the steepest slope from A, assuming that the three points are on uniformly sloping ground.

The middle reading is the mean of the other two.

Point.	Horizontal Angle.	Readings.	Angle from Vertical.
1	297° 27′	9·72 5·16	93° 56′
2	12° 18′	4·80 1·14	84° 10′

2. The readings between three stations are as follows :—

Station.	Point.	Readings.	Horizontal Angle.	Vertical Angle.
A	B	9·58 5·00	122° 45′	70° 35′
	C	7·35 2·00	75° 30′	74° 8′
B	C	6·00 2·32	176° 7′	90° 13′
	A	10·94 6·40	97° 26′	108° 39′
C	A	8·00 2·64	0° 0′	105° 50′
	B	6·70 3·00	305° 54′	89° 50′

The heights of the instrument were 4·9, 4·95, and 4·7 feet. The reduced level of A = 328·6. Find the corrected reduced levels of B and C, and plot the stations, correcting them to make them close, on a scale of 1 inch = 40 or 50 feet. The middle reading is the mean of the other two. Constant multiplier, 100. No additive constant.

CHAPTER VI.

CURVE RANGING.

General Problem.—In many cases, especially in railway surveying, it becomes necessary to connect two straight lines, AB, BC (fig. 131), on the field by a smooth curve DE which shall be tangential at both ends.

The ideal curve is a circular curve for the central portion, with a piece at each end gradually increasing in radius of curvature until it touches the straight.

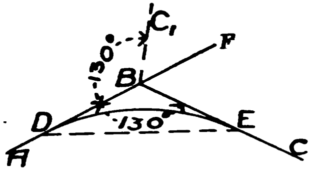


Fig. 131.

At present, however, we will assume that the curve is to be a circular arc, uniform in radius from D to E.

It is required to lay out this curve on the field, where, of course, we cannot work from the centre, and we must use some method of laying out the curve by working more or less along the arc itself.

Usually we know either the radius or the starting point of the curve, and the first step is to find whichever of the two is unknown.

The directions of the lines AB, BC, are supposed to be pegged out on the ground.

Definitions.—The point B is called the *apex* of the curve. D and E are the *tangent points*, namely the *starting* and *finishing* points.

DB = BE is called the *tangent distance*, and the straight line DE joining the tangent points is called the *long chord*.

The interior angle CBA is called the *apex angle*. For the purposes of curve ranging the apex angle is always supposed to be the interior angle. Thus if the direction of BC be altered to BC_1, the apex angle in either case would be 130°. Regarded as traverse lines with A as back station, the angle ABC_1 would be 130°, but ABC would be 230° (= 360° - 130°).

If AB be produced to F, then BF is the original direction of the route AB. But BC is the direction to which it is to be deflected by the curve. Hence FBC is called the *deflection* angle. Its value is obviously 180°—apex angle, and it may be either *right* or *left*. Thus for the line BC the deflection angle is 50° *right;* for BC_1 it is 50° *left.*

The term *deflection angle* is often loosely applied to the *tangential angles* to be afterwards referred to. The term has, however, a definite meaning in mathematics which agrees with the definition given above, and does not agree with the loose use just referred to.

Such authorities as Trautwine (see Trautwine on curves) keep the two terms entirely distinct from one another, and the writer would urge this as the only commendable course.

General Cases.—(*a*) Suppose the apex B (fig. 132) is accessible.

Then set up the theodolite at B, and measure carefully the apex angle.

$$\text{Then } \frac{\text{BD}}{\text{DO}} = \cot \text{DBO} = \cot \tfrac{1}{2} \text{ apex angle.}$$

Put DO = r, the radius.

$$\therefore \text{BD} = r \,.\, \cot \tfrac{1}{2} \text{ apex angle} \quad \ldots\ldots \quad (1)$$

$$\text{or } r = \text{BD} \,.\, \tan \tfrac{1}{2} \text{ apex angle} \quad \ldots \quad (2)$$

From equation No. 1, if r be given, we can find BD. Then set it off both ways to fix D and E. Or if D be given we can then measure BD and find r from equation No. 2. Set BE = BD to fix E.

(*b*) Suppose that B is inaccessible (fig. 133).

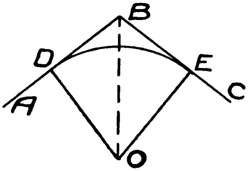


FIG. 132.

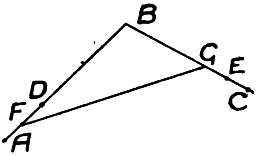


FIG. 133.

Choose any two points F and G, one in each line, such that FG can be easily measured.

Set up the theodolite at F and G in turn, and measure the angles BFG and BGF. Then

$$\underline{\text{Apex angle} = 180^\circ - (\text{BFG} + \text{BGF}).}$$

Also measure FG very carefully, and solve the triangle BFG to find BF and BG, which, of course, we cannot measure directly, as B is inaccessible.

$$\text{Thus BF} = \text{FG} \,.\, \frac{\sin \text{BGF}}{\sin \text{GBF}}$$

$$\text{and BG} = \text{FG} \,.\, \frac{\sin \text{BFG}}{\sin \text{GBF}}.$$

The angle GBF is usually over 90°, so that to find its sine from the tables we must subtract it from 180°, and look up the sine of the deflection angle, which will be the same thing.

Then, if r be given, we find BD and BE by equation No. 1 above (using the true apex angle now), and then take the difference BF − BD, and set it off from F, towards B if FB is greater than BD and *vice versa*. This fixes D, and E is fixed in exactly the same way.

If D be given, we measure FD, and note whether D is between F and B or not.

Then, accordingly, BD = BF − FD

or BD = BF + FD.

We then find r from equation No. 2 above, and GE = BE − BG = BD − BG.

This is set off in the proper direction, according to the sign, to fix E.

If D be given, it is usual to take F coinciding with D, unless we cannot find a good line DG for accurate measurement.

In finding the angles BFG and BGF we often measure the exterior angles GFA and CGF, as it may not be possible to lay out more than short lengths along FB and GB. We then subtract from 180° to find the interior angles.

The student must note that it is useless to hope for a good result when he comes to lay out the curve, unless the very greatest care is taken with these initial measurements. All lengths must be measured at any rate to the nearest tenth of a link; all angles must be repeated to see that they check, and the theodolite must be exactly centred over the station.

In setting out points, such as D and E, they should be lined in with the theodolite; and, of course, the arithmetic which has to be done on the field must be carefully checked.

The Method of Tangential Angles.—Knowing the radius, and the tangent points, we are now in a position to start laying out the curve.

There are many different methods of doing this, of which we will fully consider one only, namely by the aid of one theodolite and *tangential angles.*

Suppose DE (fig. 134) is any circular arc touching the lines DB, EB, and that DK is any chord, so that K is on the curve.

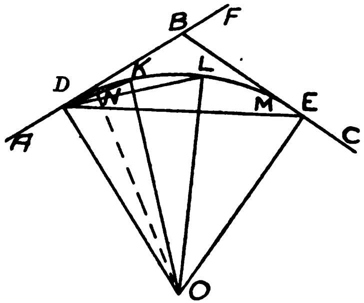


FIG. 134.

Then the angle BDK which this chord makes with the tangent DB is called the tangential angle for the chord DK. BDE is the tangential angle for the whole curve, properly called the *tangential angle for the long chord.* It is easy to prove that it is *half* the deflection angle.

Now given the radius, if we choose any length of chord DK it is easy to calculate the corresponding tangential angle.

For, by Euclid, BDK $= \frac{1}{2}$ DOK $=$ DON.

But $\sin \text{DON} = \dfrac{\text{DN}}{\text{DO}}$.

That is, $\sin \text{tangential angle} = \dfrac{\frac{1}{2}\text{ chord}}{\text{radius}}$.

The angle can therefore be found from tables, and, if we set up the theodolite at D, direct along DB with the vernier clamped at zero (making sure that the bottom plate is tightly clamped), unclamp the upper plate, and set the vernier to read the calculated angle, the theodolite will be pointing along DK.

The follower then holds one end of the selected length of chord at D, while the leader goes forward with the other end, and swings it round D as centre, as directed by the surveyor at D, until he comes into line with the cross hairs. The surveyor then gives a signal, and a peg is inserted at the point K so found. Then this is one point in the curve.

To find the next point L, if KL = DK it is evident that

$$\text{DOL} = 2 \times \text{DOK}.$$

$$\text{Hence BDL} = 2 \times \text{BDK}.$$

That is, we set the theodolite to read double the angle first found; then the follower goes to K and the leader swings the chord round that centre until he comes into line, and so on. The next angle will be three times the first.

Pegs at Ends of Chains.—Sometimes it is desired to peg out the points at once, so that they will come at the end of each complete chain from the commencement of the *line* (*not* the commencement of the curve).

If the radius is over 10 chains, the selected length of chord is nearly always 1 chain. But, if not, shorter chords, say fifty links, are sometimes chosen. The chain used may be either the Gunter's or the 100 foot.

Now D is very unlikely to come just at the end of any exact chain or half-chain, so that the first chord DK will usually be fractional. The tangential angle is calculated for the ordinary chord as before, and the angle for the fractional chord is then found by proportion. Thus if the chord be fifty links and the radius six chains,

$$\sin \text{ tangential angle} = \tfrac{25}{600} = \cdot 041667.$$

$$\therefore \text{ tangential angle} = 2^\circ\ 23\cdot 3' \text{ nearly.}$$

Now suppose the chainage of D = 23 chains + 38·4, meaning 38·4 links.

Then to make up the 50 links, the first chord will be 50 − 38·4 = 11·6 links.

The tangential angle corresponding will be $\frac{11\cdot 6}{50} \times 2^\circ\ 23\cdot 3' = \frac{11\cdot 6}{50} \times 143\cdot 3 = 33\cdot 2$ minutes, and we use this angle, and the chord 11·6, to find the first point K.

The next chord will be 50 links, and the next angle will be $33\cdot 2' + 2^\circ\ 33\cdot 3' = 3^\circ\ 6\cdot 5'$ and so on.

The angles are worked out to a decimal of a minute to avoid accumulated error, but are usually set off to the nearest minute or half minute.

Check on Finishing Point.—When the theodolite is directed towards the finishing point, the tangential angle on the theodolite should be equal to BDE. Now

$$\text{BDE} + \text{BED} = \text{BFE}.$$

$$\text{But BDE} = \text{BED}.$$

$$\therefore \text{BDE} = \tfrac{1}{2}\,\text{BFE} = \tfrac{1}{2}\text{ deflection angle}$$

$$= 90^\circ - \tfrac{1}{2}\text{ apex angle}.$$

Hence BDE can be found.

Then if the tangential angle to the point M be such that the addition of one more angle would take us beyond the calculated value of BDE, we find the extra angle required to make up the exact value of BDE, and find the corresponding length of chord by proportion as before. Then we set the calculated value of BDE on the vernier, and swing the calculated chord round M as centre till it comes into line, when we should come back on to the peg already inserted at E during the preliminary work.

Length of Curve.—The whole of the calculation above described should be worked out, checked, and tabulated *before the laying out of the points is commenced.* It will be evident that, as each chord is known, the whole length of the curve, as measured in chords, will be known also, as shown in the example which follows.

But it is a useful check on the arithmetic to calculate it independently.

There are two rules for this:—

(1) Length of curve $= \dfrac{\frac{1}{2}\text{ deflection angle}}{\text{tangential angle}} \times$ length of chord.

The angle BDE = ½ deflection angle, and gives the reading of the theodolite on the finishing point. Now for each chord, we set off the tangential angle. Hence the truth of the rule is obvious.

(2) Length of curve $= \frac{\text{deflection angle}}{360^\circ} \times 2\pi r.$

$$\text{The angle DOE} = 180^\circ - \text{EBD}$$
$$= \text{FBE}.$$
$$\text{and arc DE} : 2\pi r :: \text{DOE} : 360^\circ.$$
$$\therefore \text{arc DE} = \frac{\text{DOE}}{360^\circ} \times 2\pi r.$$

It is to be remembered that the first rule gives the length of curve as measured in chords, while the second gives the length of the actual arc. The length as calculated by the second rule should therefore be slightly longer than that obtained in the tabulation.

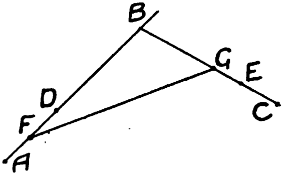


Fig. 135.

The error is about one part in two thousand if the chord be *one-tenth* of the radius.

Example of Calculation.—For example, suppose the radius is to be 15 chains, the chord 1 chain; referring to fig. 135 we find GFA = 171° 12′ 30″; EGF = 165° 19′ 30″; FG = 502·5 links, and that at the point F the chainage, from the beginning of the line, is 57·638 chains.

Then FGB = 14·40·30.
BFG = 8·47·30.

Deflection angle = 23·28·0

BG.

No.	log.
502·5	2·70114
sin BFG	9·18424
	11·88538
sin GBF	9·60012
	2·28526

BF.

No.	log.
502·5	2·70114
sin FGB	9·40370
	12·10484
sin GBF	9·60012
	2·50472

∴ BG = 192·9 and BF = 319·7 links.

$$\begin{array}{r} 180^\circ\ 0'\ 0'' \\ 23^\circ\ 28'\ 0'' \\ \hline \end{array}$$

$$\text{Apex angle} = 156^\circ\ 32'\ 0''$$

$$\tfrac{1}{2} = \quad 78^\circ\ 16'\ 0''$$

Tangent distance = r . cot $\frac{1}{2}$ apex angle

= 311·5 links = BD or BE.

∴ FD = 319·7 – 311·5 = 8·2 links, to be set off towards B, as FB is greater than FD.

And GE = 311·5 – 192·9 = 118·6 links to be set off *away* from B.

∴ chainage of starting point

= 57·638 + ·082 = 57·720 chains.

For chain chords,

sin tangential angle = $\frac{1}{30}$.

∴ tangential angle = 1° 54·6′.

First chord = 58·00 – 57·72 = ·28 chain.

∴ first angle = ·28 × 114·6 = 32·1 minutes. Then tabulate as on p. 210. The half deflection angle is 11° 44′.

After the sixth point, the angle required to make this up is 1° 38·9′, and the corresponding chord is $100 \times \dfrac{1^\circ\ 38{\cdot}9'}{1^\circ\ 54{\cdot}6'} = 86{\cdot}3$ links.

The length of curve as found by adding up column no. 2 is 614·3 links.

To check, length of curve $= \dfrac{11^\circ\ 44'}{1^\circ\ 54{\cdot}6'} \times 100$

= 614·3 links.

By second rule,

$$\text{length} = \frac{23^\circ\ 28'}{360} \times 2\pi \times 1500 = 614{\cdot}4 \text{ links.}$$

No.	Chord (Links).	Angle.	Chainage.	Remarks.
1	28	0° 32·1 1° 54·6	58·0	Curve no. 47.
2	100	2° 26·7 1° 54·6	59·0	
3	100	4° 21·3 1° 54·6	60·0	
4	100	6° 15·9 1° 54·6	61·0	Station moved.
5	100	8° 10·5 1° 54·6	62·0	
6	100	10° 5·1 1° 38·9	63·0	
7	86·3	11° 44·0	63·863	

All these calculations, etc., should be tabulated with the number of the curve attached; and when the table above is completed and checked we begin to lay out the points. As each point is fixed the angle is ticked off, so that there is no doubt as to which was last set off.

To Move Theodolite.—Now suppose that after laying down the point P, we are unable to see any farther. Let the tangential angle laid off to P be $\beta = BDP$, and draw QPR, the tangent at P.

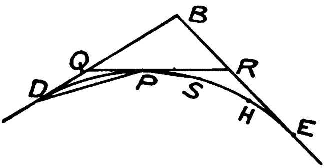


FIG. 136.

Then angle $DPQ = QDP = \beta$.

Hence the clockwise angle DPR = $180° + \beta$.

Now take the theodolite to P, set the vernier which is being used to 180°, unclamp the bottom plate, and direct back to D. See that the bottom plate is firmly clamped, and that the vernier still reads 180°. Then unclamp the upper plate, and turn the telescope until the vernier reads the *next tangential angle in the table*, after the one laid out to P. The telescope will then be in position for fixing the next point S.

We have seen that DPR = $180° + \beta$.

Hence if T be the tangential angle for 1 chord,

$$\text{DPS} = 180° + \beta + \text{T}.$$

Hence if the vernier reads 180° at D, at S it must read $180° + 180° + \beta + \text{T}$

$= 360° + \beta + \text{T} = \beta + \text{T}$, neglecting the complete revolution.

But as β was the *last* angle laid out, the next in the table will be $\beta + \text{T}$, the required reading.

Make a note in the remarks column to show at which point the theodolite was moved.

Now suppose we have to move again after laying down the point H.

Then take the theodolite to H; set the vernier to read $180° + \beta$ (β being, as before, the reading to the previous station P) and direct back to P. Then turn the telescope till the vernier reads *the next angle in the table*, after that laid out to fix H, and so on for any desired number of shifts. The proof presents no new features, and is left as an exercise.

Going back to the first shift, some surveyors prefer to set to zero (instead of 180°) on D and then to turn the telescope over vertically and set off the next angle in the tables.

This has the effect of reversing face and errors due to collimation and inclination of horizontal axis change sign, so that their effect is doubled.

On the other hand, working by the method recommended in the text, we are trusting to the graduation of the circle more than in the last method. The effect of collimation errors, etc., would be minimized however. The writer has deliberately chosen the method in the text as being that most likely to give a good result.

It differs, however, from what is given in many books.

Checking by Offsets.—Let DEF (fig. 137) be a circular arc, DB a tangent, and DE, EF equal chords.

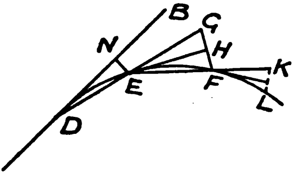


Fig. 137.

Draw EN perpendicular to DB.

Then NE = DE . sin NDE.

$$= \text{chord} \times \frac{\frac{1}{2}\,\text{chord}}{\text{radius}}$$

$$= \frac{(\text{chord})^2}{2 \times \text{radius}}.$$

DN will be nearly equal to the chord.

Thus if the leader be told the value of NE he can fix E approximately by first stretching the chord along the tangent DB, then setting off—either by eye or more exactly—the offset NE, and restretching the chord along DE. For the next point, he gets into line with D and E, and stretches the chord. This gives the point G. Now if EH be the tangent at E, FH = NE; and it is easy to show that GF = 2 × NE.

Hence if the leader pass an arrow through the ring of the tape line at G, and makes GF, as measured by tape, equal to 2NE, and at the same time stretches the full chord from E, he can thus fix the point F pretty exactly.

As a *rougher* check, he simply estimates the offset GF by eye.

Even in this way, however, he can put the arrow down within one or two inches of the correct spot as a rule; he should be trained always to do this, and if the surveyor moves him more than one or two inches, the angle is probably incorrectly set, and he should signal to that effect.

Laying Out by Offsets.—The method here described is sometimes used without the theodolite at all, for laying out the curve. But in this case we must calculate the exact length of DN for the first point. The rule is, DN = chord × cos tangential angle. The tangential angle is found by the same rule as before.

Then F is found by the two radii GF and EF from G and E as centres, as before, except that the former must be rather more carefully laid out.

All future points are checked or laid out in the same way.

It is evident (from the equation for NE) that for fractional chords the offset like NE will be proportional to the squares of the chords, but if FK be a fractional chord after the first, and K be taken in EF produced so that FK = chord FL, and if d be the full offset like NE calculated for a whole chord like EF, then the radius $KL = d \times \left(\frac{FL}{EF}\right)^2 + d \times \left(\frac{FL}{EF}\right)$ very nearly.

Approximate Rule for Tangential Angles.—It has been shown that the tangential angle for any chord is half the angle subtended at the centre by that chord.

But if the chord be not more than, say, radius ÷ ten, the chord will be practically of the same length as the corresponding arc, and we may say that the tangential angle is half the angle subtended at the centre by an *arc* equal in length to the selected chord.

$$\text{Now this angle} = \frac{\text{arc}}{\text{radius}} = \frac{\text{chord}}{\text{radius}} \text{ in radians}$$

$$= \frac{\text{chord}}{\text{radius}} \times \frac{180}{\pi} \text{ degrees}$$

$$\text{or } \frac{\text{chord}}{\text{radius}} \times \frac{180 \times 60}{\pi} \text{ minutes.}$$

$$\text{This gives tangential angle} = \frac{90 \times 60}{\pi} \times \frac{\text{chord}}{\text{radius}}$$

$$= 1718{\cdot}9 \times \frac{\text{chord}}{\text{radius}} \text{ minutes.}$$

If chord = radius ÷ ten,
tangential angle = 171·89 minutes = 2° 51′ 53″.

By the more exact formula already given the angle is 2° 51′ 58″.

It would therefore require twelve chords to be laid out before the difference would add up to one minute.

This may, however, be regarded as the limit, beyond which we should not use this rule. For chords which are shorter in proportion to the radius the error is less, and the rule is easier to work with.

Case of Left Deflection.—If the deflection be *left*, it is evident that starting at D, with the theodolite set to B as zero (fig. 138), the tangential angles will have to be set off *anticlockwise*. Hence they must be repeatedly *subtracted* starting from 360°, to find the successive vernier readings.

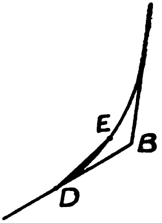


FIG. 138.

The angle to be arrived at, at the finish, is 360° – ½ deflection angle,
= 360° – 11° 44′, in the case on p. 209, and in all other respects the working is the same as before.

In this case the table, p. 210, would be as follows :—

No.	Chord (Links).	Angle.	Chainage.	Remarks.
		360° 0′ 32·1		
1	28	359° 27·9 1° 54·6	58·0	
2	100	357° 33·3 1° 54·6	59·0	
3	100	355° 38·7 1° 54·6	60·0	
4	100	353° 44·1 1° 54·6	61·0	
5	100	351° 49·5 1° 54·6	62·0	
6	100	349° 54·9 1° 38·9	63·0	
7	86·3	348° 16·0	63·863	
	614·3			

Degree Curves.—Degree curves are curves in which the tangential angle for the full chord is some exact number of degrees, or sometimes an odd half degree is introduced.

They were intended for use especially with the 100-foot chain, so that a curve of n degrees is generally defined as one in which a chord of 100 feet subtends an angle of $n°$ at the centre.

The tangential angle will then be $\frac{n°}{2}$, for a chord of 100 feet.

The radius will be $50 \times \operatorname{cosec} \frac{n°}{2}$, for sin tangential angle $= \frac{\frac{1}{2}\text{ chord}}{\text{radius}}$.

$$\therefore \sin \frac{n°}{2} = \frac{50}{r}$$

$$\text{whence } r = \frac{50}{\sin \frac{n°}{2}} = 50 \cdot \operatorname{cosec} \frac{n°}{2}.$$

All these definitions and formulæ may, of course, be applied to any other chord than 100 feet if desired.

The other calculations and methods of laying out the curves are the same as for ordinary curves. The special advantage lies in the fact that the calculation of the table of tangential angles is simplified, and, if the first chord be a full chord, all the angles, except the last, will be some exact number of degrees or half-degrees.

The radius is, however, not a round number of chains.

Transition Curves.—It has been stated that, for the best work the curves should not be simple circular arcs, but should have a piece at each end of gradually increasing radius to the straight.

These curves are called "transition curves". The best known are the *cubic parabola*, *Crandall's transition curve*, and *Searle's spirals*.

The methods of laying out these curves are, however, beyond the scope of this work, and the reader is referred to larger works on surveying for the first. The last two are described in special handbooks by Messrs. Crandall and Searle respectively.

EXERCISES FOR REVISION.

1. The bearings of two lines AB, BC, are 327° 15′ and 2° 8′ E. of N. respectively. They are to be connected by a curve of 10 chains radius, laid out in chain chords.

Find (*a*) the apex angle; (*b*) the deflection; (*c*) the tangential angle for chain chords; (*d*) the length of the curve, and (*e*) tabulate the tangential angles and chainages if the chainage of the starting-point is 37·357 chains.

2. Work the same problem if the chain is a 100-foot chain and the curve is a 6° curve.

3. A line AB is to be deflected to the left by means of a circular arc, so as to follow the direction BC, the deflection angle being 57° 12′. It is desired that this curve shall pass through a certain point X, and it is found that BX = 211·6 feet, and the clockwise angle ABX = 31° 18′. If r be the radius of the curve, and O be its centre, find the value of OB in terms of r, and then find the value of r.

4. In laying out a curve of six chains radius, by half-chain chords, the third point has been fixed, but it is found that the fourth point is invisible in consequence of an intervening obstacle. The fifth and all future points would again be visible however. Calculate the length of chord which must be used to fix number 5 point directly from number 3, and explain how you would then peg out the position of number 4, without shifting your theodolite.

5. If one peg in a curve of $n°$ has been accidentally removed,

and its position lost, prove that it can be replaced by chaining the line joining the two adjacent pegs, and at the middle point setting up an offset of 100 sin $\frac{n^\circ}{2}$.

Prove that if n be less than about 9°, this may be taken as 0·873 n without an error of more than about 1 part in 1000. The original pegs were 100 feet apart.

6. Prove in the same way as in question No. 5, that if it be desired to insert additional pegs midway between the 100-foot pegs, the offsets at the middle points of the joining chords will be 0·218 n.

7. A_1, A_2, etc., are the various apices in a proposed line. The distances and observed angles are given in the table below. The angles are measured, as in traversing, clockwise from the back station, and each distance is *from the previous apex*. A_1 is the starting-point, and the radius of each curve is given in the table below. Find the deflection and tangent distance for each curve; the whole length of line from A_1 to A_6; and the chainage of each starting and finishing point. All dimensions are in feet.

Point.	Included Angle.	Distance.	Radius.
A_2	218° 27′	876·5	1000
A_3	205° 4′	1431·8	1000
A_4	157° 42′	765·6	8° curve
A_5	153° 18′	927·8	8° curve
A_6	---	531·8	—

8. Prove that if B be the apex, A the apex angle, r the radius, and M the middle point of a circular arc, then

$$BM = r \cdot \left(\operatorname{cosec} \frac{A^\circ}{2} - 1\right).$$

Explain how you would lay out the point M, supposing the apex accessible, so as to check the curve at its middle point, and find BM for each of the curves in Q. 7.

ANSWERS TO EXERCISES.

(*Note.*—These answers have not been checked in most cases. The author will be very thankful if teachers or others who may discover errors in them will communicate with him.)

CHAPTER I. p. 60.

2. About 6 feet; 3. about 7·5 links (or feet); 5. 10·2 perches to be added; 6. (*a*) 0·7 square chain; 6. (*b*) 0·9425 square chain. (*Note.*—Work in three sections, 0 to 4, 4 to 5, and 5 to 6.)

CHAPTER II. p. 90.

2. (*a*) 321° 17′; (*b*) 139·3 N. and 111·6 W.; (*c*) 2159·3 N., 87·0 E.

3.

Point.	A.	B.	C.	D.	E.	F.
N	0	136·5	355·4	461·7	344·3	211·7
E	0	774·0	1080·3	863·0	339·2	16·7

4. 2 *a*. 2 *r*. 21 *p*.

5.

Line.	Bearing.	Length.
AB	79° 31′	774·9
BD	309° 53′	892·8

Angle ABD = 50° 22′.

6.

Point.	A.	H.	K.	B.	L.	C.	M.	N.	D.	G.	E.
N	0	790·2	708·9	232·3	766·9	268·4	972·7	468·2	63·7	367·4	294·0
E	0	0	550·4	626·5	1138·7	1211·1	2046·8	1866·3	1954·0	1196·7	1118·1

(*b*) AB = 69° 39′; DC = 285° 24′.

(*c*) HAB = 69° 39′; NDC = 297° 38′; CZB = 144° 15′; AB = 668·2; DC = 770·6.

(*d*) BZ = 325·3; CZ = 290·0.

(*e*) ZF = 193·5 = ZE; BF = 131·8; CE = 96·5.

(*f*) Co-ordinates above; bearing = 226° 55′.

(*g*) CGE = 55° 11′; GE = 107·5.

CHAPTER III. p. 152.

2. CUTTING.

Level.	Area (square feet).	Do. × Factor.
70·0	90	45
65·0	1240	1240
60·0	3380	3380
55·0	6410	6410
50·0	9250	9250
45·0	11370	11370
40·0	12970	6485
		38180

Vol. = 38180 × 5 = 190,900 cubic feet.

EMBANKMENT.

Level.	Area. (square feet).	Do. × Factor.
13·0	0	0
15·0	150	105
20·0	1530	1530
25·0	3450	3450
30·0	5690	5690
35·0	7910	7910
40·0	9530	4765
		23450

Vol. = 23450 × 5 = 117,250 cubic feet.

(*Note.*—In the embankment the first vertical interval is 2 feet, viz. from 13·0 to 15·0. The rest are 5 feet each. Hence the factor for the first area is $\frac{2}{5} \times 0{\cdot}5$, instead of 0·5, and the factor for the second area is $\frac{2}{5} \times 0{\cdot}5 + 0{\cdot}5 = 0{\cdot}75$, instead of 1·0.)

3. The average distance between contours is 25 feet about, giving an average slope of 1 in 5 for the original ground. Assuming this uniform, and taking the gradient of the centre line as 1 in 9·5, it is easy to find, by calculation or drawing, that the slope of the ground at right angles to the centre is about 1 in 5·9. With the formation width and slopes in the question, we can then calculate the sectional areas of cutting and embankment. These work out to $\frac{25}{8{\cdot}8}$ and $\frac{25}{7{\cdot}8}$ square feet respectively. Hence as there is a total length of 378 feet, we have volume of cutting = 1074 cubic feet, and volume of embankment = 1212 cubic feet.

If b_1 and b_2 be the portions of formation width in cutting and filling respectively, area of cutting $= \frac{b_1^2}{8{\cdot}8}$, and of filling $= \frac{b_2^2}{7{\cdot}8}$. Hence to equalize the volumes $b_1 = 5{\cdot}14$ feet, $b_2 = 4{\cdot}86$ feet.

4. (*Note.*—The total volume of water impounded is taken as

being up to the level of 45·0, the 6 inches between that level and 45·5 being an allowance for floods.)

WATER.

Level.	Area (square ft.).	Mean.	Volume (cubic feet).	Total Volume.	Corr. Level.
35·0	170				
37·0	2820	1495	2990	2990	37·0
39·0	8310	5565	11130	14120	39·0
41·0	18870	13590	27180	41300	41·0
43·0	34500	26685	53370	94670	43·0
44·0	41110	37805	37805	132475	44·0
45·0	46630	43870	43870	176345	45·0
			176345		

1/2 total volume = 88170 cubic feet, which occurs at the level of about 42·8 feet, where the upper valve must be placed.

DAM.

Level.	Area (square feet).	Do. × Factor.
31·0	50	25
33·0	1520	1520
35·0	4620	4620
37·0	6550	6550
39·0	6800	6800
41·0	6230	6230
43·0	5380	5380
45·0	4030	5040
48·0	1520	1140
		37305

(*Note.*—As the last vertical interval is 3 feet, while the rest are 2 feet, the factor for the last area is $\frac{3}{2}$ × ·5 = ·75 instead of ·5, and for the last but one, $\frac{3}{2}$ × ·5 + ·5 = 1·25.)

Volume of dam = 37305 × 2 = 74610 cubic feet.

Area covered by work = 59190 square feet.

5. Volume of excavation = 1840 cubic yards. Equalizing top breadth about 6 feet.

6. Curvature 0·05, refraction 0·01; curvature + refraction 0·04; level of A = 22·99; collimation error = 0·09, readings being *too low;* error for 400 feet = 0·03 nearly.

7. Widths in plan, 45·7, 51·5, 67·0, 69·4, and 74·9 feet, sectional areas of cutting in square feet = 18·5, 168, 370·5, 503, and 620; embankment, 19 square feet, volume of cutting = 89,400 cubic feet; embankment, 295 cubic feet, area of ground in plan = 16240 square feet.

Volume of cutting by trapezoidal rule = 89800 cubic feet.

CHAPTER IV. p. 180.

1. Correction per chain for temperature = ·0056
,, ,, ,, ,, tension = ·0023
,, ,, ,, ,, sag = − ·0600
− ·0521

SLOPE.

No.	h.	h^2.	$\frac{h^2}{2L}$.
1	·82	·67	·0034
2	1·21	1·46	·0073
3	1·28	1·64	·0082
4	1·06	1·12	·0056
5	·24	·06	·0003
6	·45	·20	·0010
7	·48	·23	·0012
8	·89	·79	·0040
9	1·25	1·56	·0078
10	·12	·01	·0001
11	1·24	1·54	·0077
12	·55	·30	·0007
			·0473

11·482 × ·0521 = − ·5982
Slope = − ·0473
Total = − ·6455

Corrected length

= 1148·24 – ·65 = 1147·59 feet.

2. 2′ 15″, *plus* for face left readings.

(*a*) + 2° 14′ 25″; (*b*) – 7° 52′ 5″.

3. B 5·2, A 8·6.

4. k = 1·8267; A = 117° 34′; cot BCX = 1·53858; BCX = 33° 1′ 42″; BAX = 84° 32′ 18″; ABX = 355° 56′ 18″; BX = 2118·2 feet; co-ordinates of X, 4838·2 N and 7795·1 E.

5. EE_2 = 153·7 feet nearly; E_2BE = 1° 36′ 10″, ECE_2 = 2° 9′ 10″ (each to the nearest 10″ only); BEC = 132° 20′ 40″. E_2AE = 1° 6′ 20″, AEB = 55° 29′ 50″.

7. 1147·56 feet.

CHAPTER V. p. 198.

1. Additive constant 1·4; multiplier 99·5 nearly.

Distances, 453 and 362 feet; reduced levels, 294·1 and 366·7. The steepest slope is about 1 in 6·6, in a direction making an angle of about 15½° with the line 21.

2. Distances, AB = 407·7, BA = 407·4; AC = 494·8, CA = 498·0; BC = 368·0, CB = 370·0.

Rises, AB = 141·18, AC = 140·93, CB = 0·93.

Falls, BA = 141·30, CA = 141·84, BC = 0·61.

Mean rise A to B = 141·24; A to C = 141·38; C to B = 0·77.

Hence for totals we have:—

Line.	Rise.	Corr.	Fall.	Corr.
AB	141·24	+ ·30	—	
BC	—		0·77	– ·30
CA	—		141·38	– ·31
	141·24		142·15	

The rises and falls should be equal on the closed circuit, but the table shows that the falls are too great by 0·91 (= 142·15 – 141·24).

This closing error should be distributed *not* proportionally to the rises and falls, but to the lengths of the lines. Usually the latter are taken as equal, and the closing error applied equally to each line as in the table.

Thus the corrected rise A to B = 141·24 + ·30 = 141·54; and so on. Reduced levels, A = 328·6

$$\begin{array}{rr} & 141{\cdot}54 \\ \cline{2-2} B = & 470{\cdot}14 \\ & {\cdot}47 \\ \cline{2-2} C = & 469{\cdot}67 \\ & 141{\cdot}07 \\ \cline{2-2} A = & 328{\cdot}60 \end{array}$$

CHAPTER VI. p. 217.

1. (*a*) 145° 7′; (*b*) 34° 53′ R; (*c*) 2° 51·9′; (*d*) 608·8 links.

No.	Chord (Links).	Angle.	Chainage
1	64·3	1° 50·5′ 2° 51·9′	38·0
2	100	4° 42·4′ 2° 51·9′	39·0
3	100	7° 34·3′ 2° 51·9′	40·0
4	100	10° 26·2′ 2° 51·9′	41·0
5	100	13° 18·1′ 2° 51·9′	42·0
6	100	16° 10·0′ 1° 16·5′	43·0
7	44·5	17° 26·5′	43·445
	608·8		

3. Angle ABO = 61° 24′; XBO = 30° 6′.

OB $= r$ cosec 61° 24′; and $OX^2 = OB^2 + XB^2 - 2OB \, . \, XB \, . \cos XBO$.

This gives the equation

$r^2 (\text{cosec}^2 61° 24' - 1) - 2 \times 211{\cdot}6 \times \cos 30° 6' \times \text{cosec} \, 61° 24' \times r + 211{\cdot}6^2 = 0$.

Whence r = 994 feet nearly.

4. 99·8 links; to peg out No. 4, take the middle point of this chord, and set up an offset of 2·1 links.

Point.	Deflection.	Tangent Distance.	Chainage.	
			Start.	Finish.
A_1	—	—	0	—
A_2	38° 27′ R	348·7	527·8	1198·9
A_3	25° 4′ R	222·3	2059·7	2497·2
A_4	22° 18′ L	141·3	2899·2	3178·2
A_5	26° 42′ L	170·1	3794·6	4128·6
A_6	—	—	—	4490·3

8. 59·2; 24·2; 13·8; 19·9.

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